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Summary Report
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ULTIMATE LOAD BEHAVIOR OF FULL-SCALE HIGHWAY TRUSS BRIDGES

Highway Division—Iowa Department of Transportation

ERI Project 1118S

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16. Abstract

As a result of the construction of the Saylorville Dam and Reservoir on the Des Moines River, six highway bridges crossing the river were scheduled for removal. One of these, an old pin-connected, high-truss, single-lane bridge, was selected for a comprehensive testing program which included ultimate load tests, service load tests, and a supplementary test program. A second bridge was used for a limited service load test program.

The results of the research are detailed in two interim reports. The first interim report outlines the ultimate load tests and the second interim report details the results of the service load and supplementary test program. This report presents a summary of these findings along with recommendations for implementation of the findings.

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CHAPTER 1. INTRODUCTION

The construction of the Saylorville Dam and Reservoir on the Des Moines River created an ideal opportunity to study bridge behavior. Due to the dam and reservoir construction, six highway bridges crossing the river were scheduled for removal. Five of these are old pin-connected, high-truss, single-lane bridges and are typical of many built around the turn of the century throughout Iowa and the country. Only limited information on their design and construction is available because these bridges were built circa 1900. Because there is an increasing need to determine the strength and behavior characteristics of all bridges, the removal of these five was invaluable by allowing the study of bridge behavior through testing actual prototype bridges rather than physical or mathematical models. The purpose of this testing program was to relate design and rating procedures presently used in bridge design to the observed field behavior of this type of truss bridge.

A study to determine the feasibility of performing these load tests was conducted several years ago by Iowa State University[1]. Included in the study findings was a recommendation that a broad range of programs be conducted on several of the truss bridges involved in the removal program. The first truss bridge to be replaced, the Hubby Bridge, was available for testing in June 1974. A research program was developed and undertaken by Iowa State University to conduct a number of the recommended tests. Previous reports[2,3] detailed the research and findings of the first phase of the program which covered the ultimate load behavior of the high truss bridge and the second phase of the program which covered the service load testing of the Hubby Bridge and the Chestnut Ford Bridge. The tests on the Chestnut Ford Bridge were performed while the bridge was still open to traffic. Also included in the study were several supplemental programs, including the fatigue and static testing of bars obtained from both of the above mentioned bridges. This report is a summary report on the entire project and includes an outline of the results of the program and recommendations for implementation of the findings.

Objectives

Specifications and manuals adopted by the American Association of State Highway and Transportation Officials (AASHTO)[4,5] contain criteria used in the design and rating of highway bridges in the United States. These criteria are based on rational structural analysis, actual experimental investigations, and engineering judgment. These criteria also attempt to take into account actual bridge behavior to assure safe and serviceable structures. However, as a result of the catastrophic collapse of several old bridges in the last 10 years, considerable interest has been generated in determining the actual load-carrying capacity of bridges. The load capacity of newer bridges can generally be obtained from existing plans and specifications that can be supplemented by field examinations and, if necessary, actual field tests. However, for the old pin-connected, high-truss bridges, there are generally no technical data available. There is also a complete lack of field load test data at service load levels or at ultimate load capacity. The general objective of the program was to provide data on the behavior of this bridge type in the service load range and up to ultimate capacity, as well as data on the remaining fatigue life of the tension members in the truss.

As engineers undertake the analysis and rating of these bridges, many questions arise. These include the condition of the joints, the strength of the eyes (including forgings) in the tension bars, and the behavior of the floorbeams and deck. The results reported here are limited to the two bridges tested, but the results should nevertheless provide an indication of possible answers to the questions posed above.

The specific objectives of this load test program were:

1. Relate appropriate AASHTO criteria to the actual bridge behavior as determined from tests on the available truss bridges.
2. Determine an estimate of the remaining fatigue life of the bridge components.
3. Determine the effect of repairs on the remaining fatigue life of the bridge components.

The results of the research will provide a better understanding of the actual strength of the hundreds of old high-truss bridges existing throughout Iowa as well as the country as a whole.

General Test Program

The first phase of the test program consisted of ultimate load testing of one span of the Hubby Bridge in Boone County, ultimate load testing of two I-shaped floorbeams of the bridge, and ultimate load testing of two panels of its timber deck. The truss span was tested in an "as is" condition with loads simulating actual truck loading. After initial failure the truss was damaged and retested in this condition.

The floorbeams were tested with loads to simulate an axle loading. One of the floorbeams had some initial crookedness, while the other was essentially straight. The loads were applied using hydraulic jacks and dead weights in both the truss test and the floorbeam tests. One of the timber deck tests was performed with loads simulating a truck centered on the deck panel and the other with loads placed three ft. off center to simulate a truck on the edge of the deck panel.

The second phase of the test program consisted of field service load testing of the west two spans of the Hubby Bridge and of the west span of the Chestnut Ford Bridge in Dallas County. The tests were conducted using loaded county gravel trucks to simulate a standard H truck loading. The trucks were driven along the centerline and along the edges of the roadway of each bridge.

In addition, during the removal of the two bridges, a number of tension eyebars were salvaged for use in a supplementary fatigue testing program. The laboratory tests that were conducted consisted of fatigue testing 23 eyebars in their original condition and 9 eyebars after they had been damaged and then subsequently repaired. Static tests were conducted on 19 eyebars in their original condition and on three eyebars that had been purposefully damaged in the laboratory and then subsequently repaired. Three different types of damage and repair were used which simulated the possible types of damage in the forgings and in the eyes of the bars.

CHAPTER 2. THE TEST BRIDGES

The highway bridges selected for testing were located on the Des Moines River northwest of Des Moines, Iowa, in an area which will be included in the Saylorville Reservoir. One of the high truss bridges selected was the Hubby Bridge built in 1909 (Figs. 1 and 2), located in southern Boone County about 25 miles northwest of Des Moines. It was composed of four modified Parker type high-truss simple-spans, each 165 ft. long.

The other bridge selected was the Chestnut Ford Bridge (Figs. 1 and 2), located in northern Dallas County about 20 miles northwest of Des Moines and five miles south of the Hubby Bridge. This bridge was built circa 1900 and was composed of four high-truss simple-spans. The first, third and fourth spans, from east to west, were modified Pratt-type trusses each 150 ft. long, and the second span was a Pratt truss 180 ft. long. Testing was conducted in the fourth, or west, span.

Truss Descriptions

A. Hubby Bridge

The trusses consisted of tension eyebars of both square and rectangular cross sections, built-up laced channels for the end posts and upper chord compression members, and laced channels for the other compression members. The square tension eyebars were used for truss hangers and diagonals and the rectangular tension eyebars were used for the truss lower chords and diagonals. The eyes for these two types of eyebars were formed by bending a bar around to form a tear-drop shaped eye. This tear-shaped eye was then forged to a bar to form one end of the eyebar.

The deck was built of timber stringers, timber crossbeams, and timber floor planks. The stringers stood on edge and were supported by rolled I-shaped floorbeams. A typical deck panel consisted of 15 stringers, 8 crossbeams, and 16 floor planks as shown in Fig. 3. The floorbeams were standard I-sections 12 in. deep and were connected to the truss with clip angles.

B. Chestnut Ford Bridge

The test truss consisted of tension eyebars of circular, square, or rectangular cross section for tension members, of built-up laced channels for end posts and upper chord compression members and of laced channels for the remaining compression members. One inch square tension eyebars were used for the truss hangers. Rectangular tension eyebars were used for the truss lower chords and for some of the diagonals with round bars being used for the other truss diagonals. The eyes for the square, round and the smaller rectangular eyebars were formed by bending the end of the bar around to form a tear-shaped eye and then forging it to the continuing bar. The eyes for the larger tension eyebars were machined from a plate to form a round-shaped eye and then forged to the bar.

The deck was built of timber stringers and timber floor planks. The stringers stood on edge with their longest dimension parallel to the length of the bridge and were supported by rolled I-shaped floorbeams. The cross floor planks were laminated together with bolts and were spiked to the stringers every two ft. A typical panel consisted of 13 stringers with the continuous floor planking as shown in Fig. 4.

Physical Properties

Chemical analysis and physical property tests were made of several sections from each of the bridges. The results of the analyses and tests are shown in Table 1. The tension eye-bars were determined to be made of wrought iron and the other members of steel. The timber members in the Hubby Bridge were made from Douglas Fir and pressure-treated in accordance with Iowa State Highway Commission Standards. Typical stress-strain curves for the wrought iron and steel and the load deflection curve for the timber beams can be found in the interim reports[2,3].

Table 1. Physical properties.

a. Chemical Properties

	Hubby Bridge		Chestnut Ford Bridge
Element	Percentage in Wrought Iron	Percentage in Steel	Percentage in Wrought Iron
Carbon	<0.03	0.19	<0.3
Manganese	<0.05	0.40	0.25
Phosphorus	0.29	0.012	0.130
Sulfur	0.042	0.029	0.036
Nickel	<0.05	<0.05	<0.05
Chromium	<0.05	<0.05	<0.05
Molybdenum	<0.03	<0.03	<0.03
Copper	<0.03	0.03	0.08
Aluminum	0.03	----	----
Vanadium	<0.01	----	----
Silicon	0.22	<0.05	0.12
Cobalt	0.02	----	----

Table 1 continued.

b. Material Properties

Bridge	Material	σ_y(ksi)	σ_{ult}(ksi)	E(ksi)
Hubby	Wrought Iron	35.5	49.1	28,000
	Steel	42.0	58.7	30,900
	Timber	----	4.02	1,150
Chestnut Ford	Wrought Iron	34.9	48.6	25,300
	Steel	----	----	----

CHAPTER 3. FIELD TESTS AND TEST PROCEDURE

This section summarizes the specific tests and events which occurred during the conduct of the field tests. Each testing program (i.e., timber deck test, truss test, floorbeam test, and service load tests) will be discussed separately. In this section, only the occurrences will be discussed, and the analysis of the behavior will be presented in Chapter 5.

Field Tests - Ultimate

The test procedure for each test was to

1. Apply the first load increment,
2. Hold the load until the appropriate instrumentation readings could be taken,
3. Record any behavioral indications,
4. Increase the load by the pre-established increment, and
5. Repeat steps 2-4 until failure occurs.

Timber Deck Test

The timber deck in two different panels on span 2 was the first part of the Hubby Bridge to be tested. Each of the panels was tested to failure using a simulated axle load applied by hydraulic jacks.

The first test was conducted on the panel between L_8 and L_9 with the loads centered on the panel as shown in Figs. 5 and 6. The second test was conducted on the panel between L_2 and L_3 with the loads eccentrically placed so that the center of the axle was three ft. from the center of the panel (edge wheel two ft. from edge of the roadway) as shown in Figs. 7 and 8. The tests were conducted using a self-contained system with the floorbeams acting as reactions. Instrumentation on the timber deck tests was limited to deflection dials placed across the panel mid-span between panel points. Six deflection dials were used in the first deck test, while seven were used in the second test.

The load histories of each of the two tests are given in Figs. 9 and 10. They show dramatically the effect of stringer failure as the load increased. The maximum load was 101.5 kips for the centered loading pattern and 77.4 kips for the edge loading pattern.

Truss Test

The second part of the Hubby Bridge to be tested was the trusses of span 2. The test was performed using simulated axle load applied at joints L_4 and L_5 in the ratio of 1 to 4, with the greater load being applied at L_5. This ratio was used because it represented the relationship between the axles on an AASHTO H truck.

The loads were applied using hydraulic jacks connected to large concrete mats acting as dead weights. The weights of these mats ranged from 34 kips to 112 kips. Soil was piled on top of the mats to increase their weight. Two of these mats, cast under span 2, were used for the truss test. The other two, under span 1, were used for the subsequent floorbeam tests. One inch diameter rods were attached to the mats using concrete inserts and a system of structural tubes. The hydraulic jacks were connected to the rods through a similar system of structural tubes so the loads could be applied to the truss.

Sketches of the loading system are shown in Figs. 11 and 12. The instrumentation on the truss tests consisted mainly of strain gages on the truss members.

The truss tests proceeded as planned up to a total load of 80 kips. While proceeding to a load of 90 kips the observation was made that yielding was taking place in one of the hangers at L_5 on the downstream side. The yielding made it extremely difficult to hold and increase loads. During the load increment to 110 kips, there was considerable yielding at L_5. At a total load of 110 kips, a snapping sound was heard, and the load dropped several kips; however, no visible sign of failure was evident. Loading proceeded with the same difficulty to a load of 130 kips. At this load the flaking of the rust on the hangers at L_5 (upstream side) was very noticeable.

At a total load of 133 kips (106.4 kips at L_5 and 26.6 kips at L_4), one of the hangers at L_5 (upstream side) failed. The location of the failure and a close-up of the fracture are shown in Figs. 13 and 14. Subsequent reloading to 140 kips resulted in only increased truss distortion.

It was decided that further testing of the trusses would not provide additional meaningful information. The decision was then made to pursue the objectives of the second truss test by "damaging" one of the key members and reloading. To simulate the damage, member L_2U_2 was cut with an acetylene torch. This member was damaged because it is representative of laced channel compression members.

Initial instrumentation readings were taken and reloading at only L_4 began. The load was increased to 70 kips with sets of instrumentation readings taken at periodic intervals. After a load of 70 kips was reached without any signs of additional distress, the decision was made to cut the other channel comprising member, L_2U_2, to obtain a failure of the truss. The load was again applied at L_4 with the load reaching 39 kips before the member collapsed upon itself (forming a complete but shorter member) at the cut location (Fig. 15). This resulted in a slight drop in load. The load was then increased to 72 kips with no further distress of the truss. The load was removed and all testing terminated because of potential danger of collapse during any additional member damage.

Floorbeam Test

The final portion of the ultimate test program was the testing of two floorbeams in span 1. They were both tested to failure using a load applied by hydraulic jacks and simulating a truck axle. The first test was conducted on the floorbeam at L_5. The compression flange of this floorbeam was approximately 13/16 in. out of line horizontally at mid-span. The second test was conducted on the floorbeam at L_4. The compression flange of this beam was initially straight (within allowable tolerances). The test setup and load placement on the floorbeam are shown in Figs. 16 and 17. As can be seen from these two figures, each floorbeam was loaded using a system similar to that employed for the truss test.

Instrumentation consisted of strain gages on the two floorbeams tested, as well as on the adjacent floorbeams. Strain gages were also placed on selected truss members. Deflection dials were used to measure the displacement of the test beams at the centerline and quarter points.

The first test was conducted on floorbeam 5. The load was first applied in increments of 10 kips, but as the loading progressed to higher levels the load increment was reduced to 5 kips until failure was reached. The test on floorbeam 5 proceeded as planned up to a load of 40 kips. At this load the floorbeam had started to buckle laterally between load points as well as to pull away from the timber stringers. As the load reached 45 kips the floorbeam continued to buckle laterally and pull away from the stringers. The load was then increased to 50 kips, at which point the lateral deflection due to buckling was approximately one inch beyond the initial crookedness of the floorbeam at its centerline as shown in Fig. 18. Termination of the test occurred at this point because the floorbeam was unable to sustain any further increase in load.

The test of floorbeam 4 (initially straight) proceeded without any lateral distortion or excessive end distress up to a load to 50 kips. At this load, the observation was made that the plate connecting the floorbeam to the truss was bent considerably. Loading continued up to 65 kips. After reaching this load, three bolts broke on the upstream end connection of the floorbeam to the truss. The load then dropped to 61 kips. At this time the floorbeam was approximately 3/8 in. out of line at its centerline. The floorbeam had buckled laterally only between the load points, indicating that the load points provided adequate lateral bracing. The floorbeam was then reloaded to 66 kips, when four bolts broke on the upstream connection of the floorbem to the truss, causing the load to drop to 54 kips. Further attempts to increase the load above 55 kips failed and the test was terminated due to extensive lateral buckling of the beam.

Service Load Tests

Service load tests were performed on the two west spans of the Hubby Bridge in Boone County and on the west span of the Chestnut Ford Bridge in Dallas County. The tests were accomplished using loaded gravel trucks supplied by Boone County and Dallas County. The trucks were weighed using portable scales before each test by a State Weight Officer. The weights of the trucks for each test are given in Table 2.

Table 2. Wheel loadings of trucks.

Test	Front (lbs) Left	Front (lbs) Right	Rear (lbs) Left	Rear (lbs) Right	Total (lbs)
Hubby Bridge - Span 1	3790	3780	10290	11010	28870
Hubby Bridge - Span 2	4120	3820	12500	11250	31690
Chestnut Ford	3850	3690	10260	11520	29320

The procedures used for each of the tests were the same, but the instrumentation varied. The testing procedure for each test was:

1. Take an initial reading on all instrumentation with the truck completely off the bridge,
2. Move the truck to the first desired position on the bridge,
3. Stop the truck there while readings are taken on the instrumentation,
4. Move the truck to the next desired position,
5. Repeat steps 3-4 until all desired readings have been taken, and then
6. Move the truck completely off the bridge and take a final reading of the instrumentation.

The instrumentation for Hubby Bridge span 1 consisted of 108 strain gages and five deflection dials. The deflection dials were located at the centerline, quarter points, and near the ends of the floorbeams at L_5. Of the 108 strain gages, 76 were mounted on selected truss members and 32 were mounted on floorbeams 3, 4, 5, and 6.

The strain gages on the floorbeams were mounted on the compression and tension flanges of the floorbeams. They were located at the centerline, third points, and also near the ends of the floorbeams 4 and 5 and at the centerline and near the ends of floorbeams 3 and 6. The truck was driven down the centerline of the bridge first, stopping with its rear wheels in line with the panel points. The truck was then driven down each side, with the center of the wheels approximately two ft. from the edge of the roadway, stopping only at L_3, L_4, L_5, and L_6.

The instrumentation for the Hubby Bridge span 2 test consisted of 116 strain gages and 6 deflection dials. Eight gages were mounted on the compression and tension flanges at the centerline of the floorbeams at L_2, L_3, L_8, and L_9 and the remaining 108 were mounted on the truss members.

The truck was driven down the centerline of the bridge first, stopping with its rear wheels in line with the panel points. The truck was then driven down each side stopping only at L_5 and halfway between L_2 and L_3.

The instrumentation for the Chestnut Ford Bridge consisted of 15 strain gages mounted on the north truss of the west span. The strain gages were mounted on tension members only. The truck was driven down the centerline of the bridge and then down one side of the bridge stopping at each panel point.

After this part of the test was completed the truck was located on the bridge with its rear wheels halfway between panel points. Deflection measurements of the deck were taken while the truck was at the center of the bridge roadway and at eccentric positions on the left and right sides of the bridge roadway.

CHAPTER 4. LABORATORY TESTS AND TEST PROCEDURE

The service load field tests were completed in September 1974 and the bridges were removed in January 1975. The contract for the salvage of the bridges stated that the east span of the Hubby Bridge and the west span of the Chestnut Ford Bridge were to be removed as if they were to be reconstructed. Over 100 eyebars from these spans were shipped to the laboratory.

This section outlines tests that were performed in the laboratory.

Fatigue Tests

The main thrust of the laboratory testing program was the fatigue testing of 30 eyebars. The fatigue tests were accomplished using a special apparatus design so that loads could be applied to the eyebars through pins placed in the eyes (Fig. 21). The pins used were actual pins taken from the test bridges. The pin used in the eye of an eyebar was not necessarily the one that was originally in that particular eye, but it was nevertheless a pin of the same size. The eyebars were inspected for dimensions, flaws, and peculiarities before they were tested.

The cyclic loads that were imposed on the eyebars varied from a minimum of two ksi to a maximum of 16-22 ksi. All of the fatigue tests were run with a cyclic frequency of three to four hertz.

Some of the tests were performed on undamaged bars and some of the tests were performed on bars that had been purposefully damaged in the laboratory and then repaired. Three types of damage and repair were investigated:

1. The first type of damage simulated a fracture in the forging area near a turnbuckle. Two eyebars were cut at a forging near a turnbuckle and were then welded back together. Two pieces of cold-rolled bar stock of the same dimensions as the eyebar were spliced onto the eyebar over the fracture. The splices extended for at least two ft. in each direction from the fracture.

2. The second type of damage simulated a fracture in the neck of an eye. Four bars were cut in the neck of an eye and were then welded back together. Pieces of cold-rolled bar stock were spliced on over the fracture. The splices extended as far into the eye as possible and at least two ft. along the bar past the fracture.

3. The third type of damage simulated a fracture in the eye. In this case, the eye was cut off completely and a new eye was formed out of cold-rolled bar stock. The eye was formed by heating the bar stock cherry red and bending it into a tear-shape. This new eye was then welded onto the original bar.

Static Tests

The second part of the laboratory testing program consisted of the static testing of specimens taken from 22 eyebars. The specimens were cut from the ends of the eyebars and consisted of the eye plus three ft. of the bar, except for three specimens which included only a turnbuckle section. Nineteen specimens were tested in the original condition and three specimens were tested after being repaired.

CHAPTER 5. TEST RESULTS AND ANALYSIS

In Chapters 3 and 4 a summary of the test program and the actual events which occurred during the conduct of the test were indicated. In subsequent paragraphs in this chapter the results of the test and an analysis of their significance will be presented. Each test program will be discussed separately.

Timber Deck Test

The ultimate load and equivalent H truck for each of the tests are shown in Table 3. The equivalent H truck for the deck tests was determined by placing the equivalent rear axle of the truck at mid-span of the deck panel. The total ultimate load for deck test 1 (load centered on roadway) was 101.5 kips and for deck test 2 (load placed eccentrically) it was 77.4 kips. It should be noted that although the loads were applied transversely at six-foot centers (wheel track spacing), there were two equal loads spaced longitudinally at the third-points. These loads, however, can be related to other behavior by determining the equivalent AASHTO truck. For deck test 1 (centered load), failure occurred at an equivalent H 42 truck and for test 2 (eccentric load), at a H 32 truck.

The primary behavioral indicator for the deck tests was the deflection readings taken across the width of the panel at mid-span of the panel. The load-deflection curves of the two deck tests at various points transversely across the section are shown in Figs. 22 and 23. These curves, along with the ultimate load data, indicate the behavior of the deck throughout the test to failure.

Table 3. Ultimate loads.

Test	Ult. Load (kips)	Equiv. H Truck[a]
Timber Deck		
Centered load	101.5	H 42
Edge load	77.4	H 32
Truss		
General loading	140	H 70[b]
Initial failure	133	----
Maximum load at L_4	78.5	----
Floorbeam		
At L_4	66.0	H 40
At L_5	50.0	H 30

[a] Standard AASHTO H Truck providing the same total static moment as provided by the ultimate load

[b] H 66.5 at initial fracture of L_5M_5

The behavior of deck test 1 was typical of that expected. The load deflection curves for that test (Fig. 22) indicate that the behavior of the deck up to a total load of 60 kips (H 25 truck) was linear. Beyond 60 kips the influence of stringers breaking can easily be seen in Fig. 22.

The deflection readings in Fig. 22 can be combined to form a deflection cross section at various load levels (Fig. 24). This figure gives an indication of the distribution of the load to each of the stringers. From these deflections, the amount of load distributed to each of the stringers can be calculated. The figure shows that the greatest part of the load is being carried by the stringers around and between the load points. It also indicates that the deflection increases linearly until the first stringer fails.

The percentage of the total load carried by the most heavily loaded stringer can then be compared to the distribution as determined from the AASHTO Specifications[2]. The AASHTO distribution is given as S/4 wheels in Sec. 1.3.1, where S is the average stringer spacing in feet. For deck test 1, the percentage of the total load distributed in the most heavily loaded stringer is, according to the Specifications, 14 percent.

Table 4 shows the experimental percentage of the load distributed to the most heavily loaded stringer and the equivalent distribution factor at loads below the load which caused the first stringer to fail. It can be seen that the load distribution characteristics remain the same in this case (up to stringer cracking).

Table 4. Experimental percentage of the load distributed to the most heavily loaded stringer and the equivalent distribution factor for deck test 1.

Load (kips)	Equivalent Distribution Factor[a]	Percentage of the Load Distributed to The Most Heavily Loaded Stringer
10	5.33[b]	10.5
20	5.49	10.2
30	5.38	10.4
40	5.49	10.2
50	5.49	10.2
60	5.54	10.1

[a]AASHTO = 4 from S/4 (Article 1.3.1)[2]

[b]Equivalent Distribution Factor = (14/10.5)4 = 5.33

Table 4 shows that the experimental percentages of the load distributed to the most heavily loaded stringer are less than predicted from the AASHTO Specifications. Although this loading represents the usual load case (centered loading), it should be noted that the eccentric loading (truck near roadway edge) case is more critical and will result in the edge stringers receiving more load. The Specifications cover the most critical case, and thus it would be expected that the centered load (deck test 1) would be conservative.

The theoretical capacity of the deck for deck test 1 was determined, using data from tests of stringers removed from the bridge, to be 104.7 kips. Thus, the actual capacity of the deck (101.5 kips) is very close to the theoretical capacity.

The behavior of deck test 2 (eccentric loading) was also typical of that expected. The load-deflection curves for that test (Fig. 23) indicate that the behavior of the deck was linear up to a total load of 40 kips (H 17 truck). The behavior of the deck shown by Figs. 23a and 23b is not really indicative of behavior of the entire deck because these two deflection dials were near the edge of the panel opposite the loading. This portion of the deck underwent only uplift and very small deflections.

The deflection readings in Fig. 23 are combined in the same manner as Fig. 22 to form a deflection cross section at various loads (Fig. 25). Figure 25 gives an indication of the distribution of the load to each of the stringers. This figure also indicates that the major portion of the load is being carried by the stringers on the loaded side of the panel and that the deflection increases linearly up to a total load of 40 kips.

As in deck test 1, the percentage of the total load carried by the most heavily loaded stringer can be compared to the distribution as determined by the Specifications[2]. For deck test 2 the percentage of the total load distributed to the most heavily loaded stringer is about 15 percent at the equivalent of an H 15 truck. Table 5 shows the experimental percentage of the load distributed to the most heavily loaded stringer at loads below the load which caused the first stringer to fail.

Table 5. Experimental percentage of the load distributed to the most heavily loaded stringer and the equivalent distribution factor for deck test 2.

Load (kips)	Equivalent Distribution Factor[a]	Percentage of the Load Distributed to The Most Heavily Loaded Stringer
10	4.00[b]	13.7
20	3.69	14.9
30	3.48	15.8
40	3.62	15.2

[a]AASHTO = 4 from S/4 (Article 1.3.1)[2]

[b]Equivalent Distribution Factor = (13.7/13.7)4 = 4.00

Table 5 indicates that the experimental percentages of the load distributed to the most heavily loaded stringer are equal to or slightly greater than those predicted by the AASHTO Specifications[2] (13.7 percent). It would be expected that the critical stringer (at edge) would carry a higher percentage of the load for this more severe eccentric case than in the centered case (test 1).

Table 5 also indicates that the distribution did change slightly as the load increased. This could be attributed to a very high moment gradient in the weaker transverse planking, which is the major distributing agent.

The theoretical capacity of the deck for deck test 2 was determined to be 78.5 kips. This is extremely close to the actual capacity of the deck (77.4 kips).

The results from both deck tests indicate a high degree of validity for both the distribution procedure indicated by AASHTO[2] and the calculations for deck capacity. It should be noted, however, that the timber deck used in the bridge consisted of heavy transverse planks to assist distribution. Distribution characteristics could vary significantly for other deck types. Thus, although there is a good comparison in this case, there is a possibility of need for consideration of various deck configurations in distribution determination.

Truss Test

The initial failure of the truss took place at a load of 133 kips. This failure was the breaking of one of the hangers which made up member L_5M_5. The applied loading was 106 kips and 27 kips at L_5 and L_4, respectively. Additional load was applied in an attempt to cause additional members to fail. A large distortion of the lower chord of the truss near the load at L_5 occurred under this higher loading without any failure. The maximum load under this general loading was 140 kips; 112 kips at L_5 and 28 kips at L_4. The maximum vertical deflection at L_5 at this time was 15 in.

After adjustment of the loading system, all load was applied at L_4 with the maximum load being 78.5 kips. The test program then included damaging a member. After member L_2U_2 was cut completely through, a load of 39 kips produced a failure of the truss. This resulted in a vertical displacement of the member at the cut location.

The behavioral indicators for the truss test were the deflection readings at mid-span and at the three-tenths points and the forces in the truss members as computed from the strain gage readings taken during the test. The experimental strains were converted to stresses assuming that both the wrought iron and steel were elastic-perfectly plastic materials. The materials were assumed elastic up to the yield strain computed from appropriate values of yield stress and modulus of elasticity in Table 1 and assuming no increase in stress beyond the yield strain. The areas of each individual member were used to convert the stresses to forces in the individual members. Figure 26, the theoretical and experimental load-deflection curves for the vertical deflection at mid-span, indicates that yielding began to occur in member L_5M_5 at a total load of approximately 80 kips. The curve was relatively linear at loads less than 80 kips and above 80 kips the slope of the curve decreases, indicating yielding of member L_5M_5. The theoretical and experimental load-deflection curves for the vertical deflection at L_3 and L_7 also indicate no yielding or nonlinearity up to the maximum load at which readings were taken.

Figure 27, the total load-force in truss member L_5M_5 curve, indicates, for this truss, approximately the same behavior as the total load-vertical deflection curve at L_5 (Fig. 26). Curves that illustrate total load-force in other truss members also indicate linear behavior up to the maximum load at which readings were taken.

The theoretical forces used in Figs. 26 and 27 were obtained from a structural analysis of the truss assuming that all of the members were held together by pins at the joints. Most of the experimental forces determined from strain gage readings agree quite closely with the theoretical forces determined from analysis. Some of the experimental data for the vertical members is erratic or differs considerably in magnitude from the theoretical curve, but the slope of the curve is very similar to that of the theoretical curve. This behavior is due to the "frozen" condition of the truss joints resulting from the rusted members and pins.

Thus, although the actual conditions in the joints are unknown, considering the truss to be pin-connected does provide a realistic method of truss analysis for these old bridges. The tremendous flexibility of the members that allows accommodation of any joint restraint contributes to this conclusion.

The capacity of the hangers at L_5 as calculated using data from coupon tests was 110 kips. This was just a few kips greater than the load that actually caused the fracture of one of these hangers. The actual stress at fracture was 47.4 kips/square in. This indicates that the "lap," near where the fracture occurred, was about 97 percent effective. An examination of the fracture (Fig. 14) indicates also that only a very small portion of the section was not fused. The current practice is to assume the "lap" only 40 percent effective, which is much lower than the actual capacity of the member.

Floorbeam Test

The maximum load applied to the floorbeam at L_4 was 66.0 kips. The compression flange of this floorbeam was originally straight (within allowable tolerances). The maximum load applied to the floorbeam at L_5 was only 50.0 kips, but this floorbeam had an initial crookedness of approximately 13/16 in.

The primary behavioral indicators for the floorbeam tests were the vertical deflections of the floorbeam along its length and the moments on the floorbeam as computed from strain gage data.

The load-deflection curves for the floorbeam test at L_4 are shown in Fig. 28 and indicate that a departure from linearity occurs at a load of about 40 kips (H 24 truck). At this same load the observation was made that the floorbeam was beginning to buckle laterally. This indicates that the natural dapping of the stringers provides sufficient lateral support of the floorbeam up to about 60 percent of the ultimate load. Beyond 60 percent of the ultimate load the floorbeam buckled laterally between the load points and deflected away from the stringers between the load points because there was no positive tie between the stringers and the floorbeam.

The load-deflection curves for the floorbeam test at L_5 are shown in Fig. 29 and indicate a departure from linearity at a load of about 35 kips (H 21 truck). At about that load the observation was made that the floorbeam was beginning to buckle laterally. This departure from linearity thus gave an indication of the initiation of lateral buckling in the floorbeam and again shows that the natural dapping of the stringers provides sufficient lateral support of the floorbeam up to about 70 percent of the ultimate load. Beyond 70 percent of the ultimate load the floorbeam buckled laterally between the load points due to the lack of a positive tie between the stringers and the floorbeam.

The theoretical capacity of the floorbeam (initially straight) was calculated at 62.4 kips. This was based on the assumption that the load was uniformly distributed to the floorbeam and that the ends were partially fixed. This agrees quite closely with the actual capacity of the floorbeam (65 kips) that was initially straight (within allowable tolerances). The theoretical capacity of the floorbeam (initially crooked) will be somewhat less than that of the initially straight floorbeam. Thus, the actual capacity of the initially crooked floorbeam will agree quite closely with its theoretical capacity.

The final configuration of each of the floorbeams was evidenced by a large amount of lateral buckling of the floorbeam, as was anticipated. The compression flanges of each floorbeam were tilted and severely deformed (Fig. 18). The floorbeam had also pulled away from the timber stringers above it.

Rating

One of the significant portions of this study was the rating of the test span (span 2) and the comparison of that rating with the actual capacity.

The field inspection used as the basis for the rating calculations was made by the Maintenance Department of the Iowa State Highway Commission. This information was forwarded to the agencies cooperating in this phase of the study. These agencies were the U.S. Army - Corps of Engineers, the Highway Division, Iowa Dept. of Transportation and Iowa State University. Using this data as a base, each agency computed the rating of the bridge using the AASHTO Maintenance Manual[5].

Ratings were requested for each of the three separate portions of the truss tested, i.e., the deck, the floorbeams, and the trusses. The results of the ratings are shown in Table 6.

Ratings were requested for each of the three separate portions of the truss tested, i.e., the deck, the floorbeams, and the trusses. The results of the ratings are shown in Table 6.

Table 6. Bridge ratings (operating).

Bridge Portion	Agency			Test Capacity (Table 2)
	1	2	3	
Deck	H 13.1	H 8.2	H 9.4	H 32
Floorbeam	H 2.4[a]	H 7.4	H 6.7	H 30
Truss	H 11.4	H 12.7	H 11.9	H 66.5[b]

[a]Did not consider beam laterally supported

[b]Initial fracture of L_5M_5

It can be seen that the ratings are quite consistent for the truss. However, there is a variation in the ratings for the floor system. In the case of the floorbeams, the assumptions related to lateral support of the compression flange are critical. Table 6 shows the effect of this assumption in the rating of the floorbeam.

Also shown in Table 6 are the capacities as determined from the field tests. It can be seen that the critical member as determined by the ratings (floorbeam) is also the critical member as found from the tests.

The relationship of the ratings at operating levels to the ultimate capacity range from ratings of only seven percent of ultimate capacity for the floorbeam (assuming no lateral support) to about 40 percent for the deck. Except for the one floorbeam rating, the ratings are about 25 percent of capacity. However, it should be noted that the Manual[5] used for rating indicates a yield point of 30 ksi for steel made at the time of construction of the Hubby Bridge, whereas the actual yield strength of the steel and wrought iron was 42 ksi and 35 ksi, respectively. Since the ratings do consider dynamic effects and minimum material properties and are at the higher level (operating), the ratings appear to be quite conservative.

The results do, however, emphasize the need to accurately determine the real lateral support conditions for the beam, the realistic load distribution in the deck, and the actual material properties. Although, in this case, there were no positive supports, the natural dapping of the stringers did provide this lateral support.

Service Load Tests - Trusses

Figures 30-32 illustrate typical experimental and theoretical influence lines obtained from the results of the service load tests for truss members of the Hubby Bridge and Chestnut Ford Bridge.

The experimental influence lines were found by calculating the forces in the bar using the strain measurements that were recorded for each position of the truck. The theoretical influence lines were determined by placing a theoretical truck of the same configuration as the experimental truck (Figs. 19 and 20), at each panel point and calculating the resultant bar force using determinate analysis. Each of the graphs shows the theoretical influence line for the member as a solid line. In the testing of the two spans of the Hubby Bridge, both the north and the south truss were instrumented. The experimental influence lines for both trusses are shown as broken lines. Only the influence lines for a truck on the centerline of the bridge are shown. In the testing of the one span of the Chestnut Ford Bridge only the north truss was instrumented. The experimental influence lines are shown as broken lines.

The results showed that in most cases for the Hubby Bridge and in all cases for the Chestnut Ford Bridge, the experimental results agree closely with the theoretical values. The general shape of the experimental influence line is the same as the shape of the theoretical influence line although the magnitude of the experimental values is less than the magnitude of the theoretical values. This difference is due in part to the partial continuity of the deck which was not taken into account in the theoretical analysis, the condition of the joints, as well as problems in the instrumentation.

In the service load tests of both spans of the Hubby Bridge the multiple-channel data acquisition system was not available due to technical problems and, thus, the strain measurements were taken using older equipment. This required a longer time period and meant that variances in the power line voltage to the strain indicators, indicator drift, and the changing temperature in the bridge members occurred. These changes had an indeterminable effect on the strain measurements and resulted in unusual behavior in several members.

In addition to the recording of member strains during the service load testing of the Hubby Bridge, truss deflections were also recorded. The experimental deflections were measured during the test with the truck at each panel point. The theoretical deflections were determined from an analysis of the truss treated as an ideal pin-connected truss. It was found that the experimental deflections are much lower than the theoretical deflections. This is due to the partial continuity of the deck, which was not taken into account in the theoretical analysis, and the frozen conditions of many of the pin-connections.

Thus it appears that the analysis of a pin-connected truss, even though the condition of the pins is unknown, as a simple determinate truss will provide a conservative indication of the bar forces and truss deflections. Similar results were found during the static ultimate load tests conducted on the Hubby Bridge and reported in the first interim report.[2]

Service Load Tests - Floorbeams and Timber Deck

Figure 33 shows the experimental moment diagram for the floorbeams at L_3, L_4, L_5, and L_6 compared with the theoretical moment diagrams with the truck placed on the centerline and edges of the bridge. The experimental moments were determined from strain gages mounted on the floorbeams. The experimental moments fall between the theoretical values for fixed ends and pinned ends. The experimental moment diagrams for the floorbeams at L_3 and L_5 tend to agree more closely with the theoretical fixed end moments, while the experimental moment diagrams for the floorbeams at L_4 and L_6 tend to agree more closely with the theoretical fixed end moments. This shows the difference in stiffness of the two different types of joints. These results agree with the results reported in the first interim report.[2] The results here also show the excellent distribution properties of the deck.

For the deck sections, the experimental deflections of the stringers were compared with the theoretical deflections obtained assuming the stringers to be fixed or pinned at the far ends. In all of the cases the experimental deflections were close to the theoretical values for stringers with pinned ends, however, when the gross deflections are large, as in the case with the truck on the edge, the experimental values move away from the values for the theoretical pinned-end condition and toward the theoretical values for the fixed end assumption. This shows that when the deflections of the deck become large the load distribution characteristics improve due to the improved effects of the layered deck.

The load distribution characteristics of the bridge deck can be found approximately by using the deflection readings taken during the service load testing. The AASHTO specifications for load distribution states that the load to be taken by each stringer is found using the equation S/D where S is the stringer spacing for the deck in feet and D is given as 4 for the Hubby Bridge deck and 4.5 for the Chestnut Ford Bridge deck.

Table 7 lists the experimental values of D found for the deck tests on the Hubby and Chestnut Ford Bridges. Table 7 also lists the percentage of the total load carried by the most heavily loaded stringer. From this table it can be seen that the AASHTO specifications are conservative for the timber deck system used on the Hubby Bridge and nonconservative for the eccentric truck on the Chestnut Ford Bridge.

However, it should be noted that the maximum deflection of the critical stringer was essentially the same for both load cases on the Chestnut Ford Bridge indicating the maximum beam moment was the same.

Table 7. Load distribution factors.

Test	Equivalent Distribution Factor	Percentage of the Load Distributed to the Most Heavily Loaded Stringer
Hubby Bridge		
Truck in Center	5.83	9.6%
Truck on Left	5.71	9.8%
Truck on Right	5.77	9.7%
Chestnut Ford Bridge		
Truck in Center	4.52*	13.6%
Truck on Left	3.24*	19.0%

*Maximum deflection of critical stringer the same in both cases.

Fatigue Tests

Fatigue tests were performed on 26 tension eyebars taken from the Hubby Bridge and four tension eyebars taken from the Chestnut Ford Bridge. Some of the eyebars received at the laboratory had kinks and bends in them that were formed during the dismantling of the bridges. However, these bars were straightened on a rebar bender before testing. The residual stresses induced in the eyebars due to the straightening had no apparent effect on their fatigue life. This is a reasonable assumption since no failures occurred at the points of bending.

The eyebars tested were all of square cross section and varied from 3/4 in. to 1-1/8 in. in dimension.

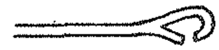
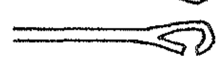
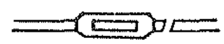
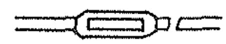
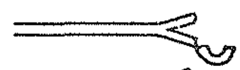
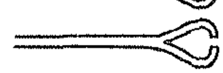
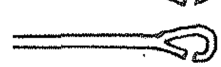
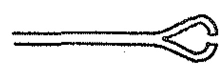
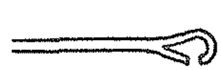
Twenty-three of the eyebars were tested in their undamaged (except for straightening) condition. The maximum stress for the tests varied from 16 to 24 ksi with a uniform minimum stress of two ksi. All of the eyebars were tested at a cyclic rate of three-four hertz. The results of the fatigue tests on undamaged eyebars can be found in Table 8. This table lists the identification number of the eyebar, its location on the truss, the dimensions of the bars, the stress range that the bar was subjected to, the number of cycles required to fail the eyebar, and a diagram illustrating the location of the failure.

Two of the 23 undamaged eyebars fractured in one of the forgings joining the turnbuckle to the eyebar. These two eyebars initially had large cracks at the point of fracture prior to the beginning of the fatigue tests.

The remaining 21 eyebars each fractured in one of the eyes of the eyebar. The fractures in the eyes occurred in two different places: 1) at the tip, and 2) at the side of the eye. A careful study of Table 8 will show that different bar sizes generally behaved in the same way except for the 7/8 in. bars which all failed at significantly lower number of stress cycles. This table also shows that the two fractures near the turnbuckles occurred at much lower numbers of stress cycles than did the fractures in the eyes.

It is assumed in the inspection and rating of bridges that the critical section of the eyebar is the section at a forging, where many small cracks exist. Since it is impossible to determine the extent of these cracks by inspection, consultants in Iowa usually assume, for rating purposes, that there is a reduction in strength

Table 8. Results of fatigue tests on undamaged eyebars.

Identification Number*	Member	Dimensions	Stress Range (ksi)	Number of Cycles	Location of Fracture
H3	L4M3	1 1/8" x 1 1/8"	14	1,415,200	
H6	L4M3	1 1/8" x 1 1/8"	16	446,180	
H5	L4M3	1 1/8" x 1 1/8"	18	121,610	
$H16_1$**	U4M5	1" x 1"	14	2,033,250+	
H9	U4M5	1" x 1"	16	787,410	
H10	U4M5	1" x 1"	16	371,950	
C32	-----	1" x 1"	16	500,450	
$H16_2$**	U4M5	1" x 1"	18	63,040	
H8	U4M5	1" x 1"	18	70,570	
C31	-----	1" x 1"	18	154,960	
H1	U4M3	1" x 1"	20	102,210	
H18	U4M5	1" x 1"	20	127,320	
C30	-----	1" x 1"	20	173,330	
C29	-----	1" x 1"	22	63,220	
H12	L4M5	7/8" x 7/8"	14	99,200	
H20	L4M5	7/8" x 7/8"	15.5	112,750	
H13	L4M5	7/8" x 7/8"	16	106,100	
H15	L4M5	7/8" x 7/8"	16	165,280	
H14	L4M5	7/8" x 7/8"	20	74,790	
H19	L4M5	7/8" x 7/8"	20	94,440	
H23	L1U1	3/4" x 3/4"	16	314,950	
$H24_1$**	L5M5	3/4" x 3/4"	16	329,990	
$H25_1$**	L3M3	3/4" x 3/4"	16	314,310	
H28	L3M3	3/4" x 3/4"	16	510,200	

* Prefix C indicates that the eyebar came from the Chestnut Ford Bridge. H indicates Hubby Bridge.

** Subscript indicates the order of the tests on a single eyebar.

of the bar of up to 60 percent. In other words, the forging is assumed to be only 40 percent of the strength of the bar.

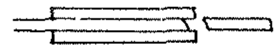
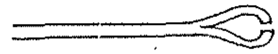
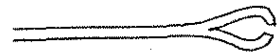
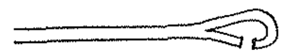
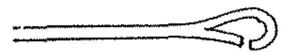
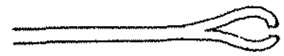
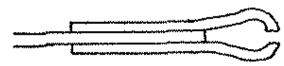
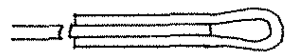
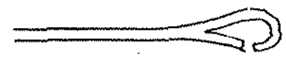
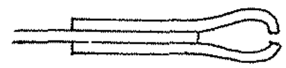
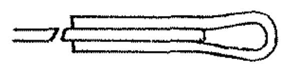
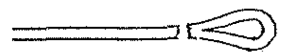
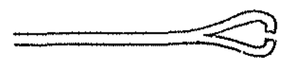
In the fatigue tests it was found that the forgings are usually not the critical points for fracture. Twenty-one of the 23 eyebars tested fractured in the eyes and not in the forgings. This indicates that the repeated flexing occurring in the eyes is the critical factor determining the remaining fatigue strength of the bar.

Fatigue tests were performed on nine eyebars taken from the Hubby Bridge in order to determine the effect, if any, of repairs on their fatigue life. The minimum stress and maximum stress for all of the tests were two ksi and 18 ksi, respectively (stress range of 16 ksi). All of the tests were run at cyclic rate of three-four hertz. The results of the fatigue tests on these damaged and repaired eyebars are shown in Table 9.

One of the nine damaged and repaired eyebars tested was a bar that was damaged and repaired at the bridge site an estimated 40 years ago (Table 9; H 4). The bar had fractured at the forging connecting the eye to the bar and the repair consisted of welding the pieces back together with two additional splice bars (one on each side). The design of the repair was inadequate since the splice did not extend very far onto the eye. In addition, the weld was of very poor quality with very little penetration into the base metal. The fatigue failure occurred at the point of repair.

Six of the eyebars were damaged and repaired in the laboratory. Four of these simulated fractures near an eye and two simulated fractures near a turnbuckle. The methods of repair for these fractures were given in Chapter 4. These repairs proved to be at least as

Table 9. Results of fatigue tests on damaged and repaired eyebars.

Identification Number	Type of Repair **	Member	Dimensions	Stress Range (ksi)	Number of Cycles	Location of Fracture	
H4	2	L_4M_3	1-1/8" x 1-1/8"	16	109,370	††	
H2	2	U_4M_3	1" x 1"	16	295,860	†	
H7	2	U_4M_5	1" x 1"	16	319,550	†	
H11	1	U_4M_5	1" x 1"	16	450,840	†	
H17	1	L_4M_5	7/8" x 7/8"	16	130,870	†	
$H24_2$*	3	L_5M_5	3/4" x 3/4"	16	626,130	†	
$H24_3$*	3	L_5M_5	3/4" x 3/4"	16	398,660		
$H24_4$*	3	L_5M_5	3/4" x 3/4"	16	1,152,560		
$H25_2$*	3	L_3M_3	3/4" x 3/4"	16	537,850	†	
$H25_3$*	3	L_3M_3	3/4" x 3/4"	16	99,880		
$H25_4$*	3	L_3M_3	3/4" x 3/4"	16	1,791,840		
H26	2	L_3M_3	3/4" x 3/4"	16	243,960	†	
H27	2	L_3M_3	3/4" x 3/4"	16	242,200	†	

* Subscript indicates the order of tests on an eyebar.

** 1 indicates damage and repair to a forging near a turnbuckle.
2 indicates damage and repair to a forging at an eye.
3 indicates damage and repair to an eye.

† The fracture did not occur near the repair.

††This member was damaged and repaired in the field.

strong as the bars since no failures occurred near the repairs. Upon testing, five of the eyebars fractured in the eyes, and one of the eyebars fractured in the forging near the eye at the end opposite from the repaired end.

Table 10 shows the results of the tests of damaged and repaired eyebars. It can be seen from this table that only two eyebars repaired in the laboratory fractured due to the presence of a weld. These fractures occurred at well over 1,000,000 cycles (many more than could be expected in a normal remaining bridge life) in eyebars that had been repaired three times. Thus, any of these repair methods appears to be appropriate for field use. Care, however, should be taken to provide good quality welding.

Static Tests

Static tests were performed on 17 specimens from eyebars taken from the Hubby Bridge and the Chestnut Ford Bridge. The specimens consisted of an eye plus two-four ft. of bar. The bars were of square, round, and rectangular cross section with seven, four, and six bars of each size tested, respectively. In addition to these specimens, two static tests were conducted on specimens consisting of round bars with turnbuckles. The results of the static tests are shown in Table 10.

As can be seen in Table 10 all of the round eyebars, including the two specimens with turnbuckles, fractured in the bars and not in the eyes or forgings. Of the seven square eyebars tested, four fractured in the bar and three fractured in the forgings. All of the rectangular eyebars fractured in the forgings.

Table 10. Results of static tests on undamaged eyebars.

Identification Number	Dimensions	Yield Stress (ksi)	Ultimate Stress (ksi)	Location of Fracture
H21	7/8" x 7/8"	32.7	58.4	
A	3/4" x 3/4"	29.7	44.7	
H	3/4" x 3/4"	31.9	43.9	
B	1" x 1"	35.0	48.0	
D	1" x 1"	36.4	46.1	
E	1" x 1"	33.3	50.2	
F	1" x 1"	36.5	47.0	
I	2.1" x .8"	36.1	49.4	
J	2.1" x .8"	34.6	45.1	
K	2.1" x .8"	35.1	46.8	
L	2.1" x .8"	----	28.9	
M	2.1" x .8"	37.6	38.7	
T	2.1" x .8"	33.9	47.1	
N	7/8" Dia	35.9	50.4	
P	7/8" Dia	35.1	50.3	
Q	7/8" Dia	37.7	48.4	
S	7/8" Dia	33.4	50.0	
O	7/8" Dia	35.3	50.9	
R	7/8" Dia	35.5	44.6	

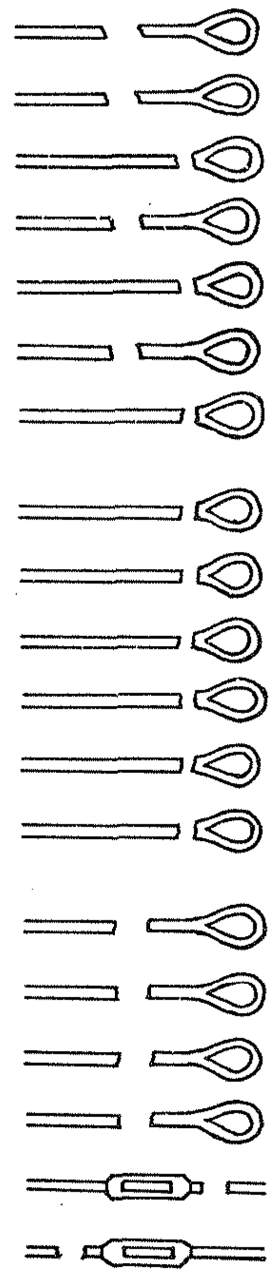


Table 11 shows the average yield and ultimate stresses for the different shapes of eyebars and the different locations for the fractures.

Table 11. Summary - static test results.

Type of Eyebar	Location of Fracture	Average Yield Stress	Range In Ultimate Stress	Average Ultimate Stress
Round	bar	35.5 ksi	44.6-50.0	49.1 ksi
Rectangular	forging	35.5 ksi	28.9-49.4	42.7 ksi
Square	bar	32.7 ksi	44.7-58.4	50.3 ksi
Square	forging	34.5 ksi	43.9-47.0	45.7 ksi

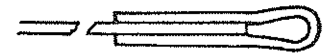
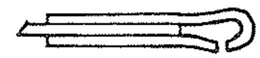
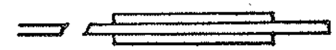
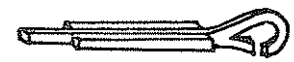
It can be seen from Table 11 that the average yield stress was approximately the same for all of the eyebars. The average ultimate stress, however, varied for the different types of eyebars. The average ultimate stress for the square eyebars that fractured in the forgings was almost five ksi less than the average ultimate stress for the square eyebars that fractured in the bar away from any forgings. Thus, the forgings that fractured in the square bars were 93 percent effective on the average with a lower bound of 90 percent. The forgings in the rectangular bars were 87 percent effective on the average with a lower bound of 59 percent.

Static tests were also performed on three damaged and repaired specimens of square cross section. Each of the specimens was damaged and repaired by one of the methods described in Chapter 4. One of the specimens simulated a fracture and repair at the forging near a turnbuckle.

The results of the static tests on damaged and repaired eyebars are shown in Table 12. It can be seen from this table that if repairs to damaged bars are made similarly to those used in these tests, then the ultimate strength of the bar will be unaffected by the repair.

The ultimate strength of the bar is slightly less than that listed in Table 1 because the stress in the eyebars was calculated using the gross cross section of the bar. This shows that after several years of rusting and corroding, the bars are still a nominal 94 percent effective.

Table 12. Results of static tests on damaged and repaired eyebars.

Identification Number	Type of Repair*	Dimensions	Yield Stress (ksi)	Ultimate Stress (ksi)	Ultimate Force (kips)	Location of Fracture
U_1	3	3/4"x3/4"	32.1	47.3	26.5	
U_2	3	3/4"x3/4"	--	--	64.6	
V	1	1"x1"	33.8	48.5	48.5	
W_1	2	1"x1"	36.8	48.0	48.0	
W_2	2	1"x1"	--	--	83.0	

*1 indicates damage and repair to a forging near a turnbuckle
2 indicates damage and repair to a forging at an eye
3 indicates damage and repair to an eye

CHAPTER 6. SUMMARY

As a result of the construction of the Saylorville Dam and Reservoir on the Des Moines River, six highway bridges were scheduled for removal. Two of these, old high-truss pin-connected single-lane bridges, were selected for a testing program which included ultimate and service load tests in the field and fatigue and static tests on tension eyebars in the laboratory.

Ultimate Load Tests

The purpose of the ultimate load tests was to relate design and rating procedures presently used in bridge design to the field behavior of this type of truss bridge. The general objective of the test program was to provide data on the behavior of this bridge type in the overload range up to collapse.

The information available on overload and ultimate behavior of actual bridges is limited mainly to beam-and-slab type bridges. No information is available on the behavior of the old high-truss bridges typical of those found in Iowa and throughout other parts of the country. This load test program is intended to provide that information on the ultimate load carrying capability through the testing of a typical old truss bridge.

The test program consisted of ultimate load testing of one span of the bridge, ultimate load testing of two I-shaped floorbeams, and ultimate load testing of two panels of the timber deck. The truss span was tested in an "as is" condition with loads simulating actual truck loading. After initial failure the truss was damaged and retested in this condition. The floorbeams were tested with loads to simulate an axle loading. One of the floorbeams had some initial crookedness, while the other was essentially straight. One of the timber deck tests was performed with loads simulating a truck centered on the deck panel and the other with loads placed three ft. off center to simulate a truck on the edge of the deck panel.

The total ultimate load for deck test 1 (load centered on roadway) was 101.5 kips and for deck test 2 (load placed eccentrically) it was 77.4 kips. For deck test 1 this is equivalent to a load of 25.4 kips at each of the load points, with the corresponding maximum moment on the total deck panel at 279.4 ft-kips or 17.5 ft-kips per foot of width of the deck panel. For deck test 2 the equivalent load and moments are 19.4 kips, 212.8 ft-kips, and 13.3 ft-kips per foot of width, respectively. It should be noted that although the loads were applied transversely at 6-foot centers (wheel track spacing), there were two equal loads spaced longitudinally at the third-points. The loads, however, can be related to other behavior by determining the equivalent AASHTO H truck. For deck test 1 (centered load) failure occurred at an equivalent H 42 truck and for test 2 (eccentric load) at a H 32 truck.

The behavior of the deck at loads up to failure of one of the stringers compared quite well with that predicted by the AASHTO Specifications[2]. The current load distribution criteria indicate that each stringer should be designed for about 14 percent of the total load on the bridge. The test results gave only about 10 percent for a centered load, but for the eccentric severe loading, the most heavily loaded stringer carried about 15 percent of the total load.

The initial failure of the truss took place at a load of 133 kips. This failure was the breaking of one of the hangers which made up member L_5M_5. The applied loading was 106 kips and 27 kips at L_5 and L_4, respectively. Additional load was applied in an attempt to get additional members to fail. A large distortion of the lower chord of the truss near the load at L_5 occurred under this higher loading without any failure. The maximum load under this general loading was 140 kips (H 70 truck), 112 kips at L_5, and 28 kips at L_4. The maximum vertical deflection at L_5 at this time was 15 inches.

The fracture load for the vertical failure was 97 percent of the calculated load based on the full section. The fracture section confirmed that the section was nearly fully fused. This compares to the "40 percent effective" used by many designers in evaluating structures of this type.

After adjustment of the loading system, all load was applied at L_4 with the maximum load being 78.5 kips. The test program then included damaging a member. After member L_2U_2 was cut completely through, a load of 39 kips produced failure of the truss. This resulted in a vertical displacement of the member at the cut location.

The maximum load applied to the floorbeam at L_4 was 66.0 kips. The compression flange of this floorbeam was originally straight (within allowable tolerances). This load was approximately equal to that determined from theory.

The maximum load applied to the floorbeam at L_5 was 50.0 kips. This floorbeam had an initial crookedness of approximately 13/16 in.

Service Load and Supplementary Tests

The purpose of the service load tests was to relate design and rating procedures presently used to the field behavior of this type of truss bridge. Another objective of this phase of the program was to provide data on the behavior of this bridge type in the service load range and also, data on the remaining fatigue life of the tension members in the truss.

The information available on service-load behavior of actual bridges is limited mainly to beam-and-slab type bridges. This test program was intended to provide information on the behavior of high-truss bridges.

The test program consisted of service load testing two spans of the Hubby Bridge plus one span of the Chestnut Ford Bridge, and fatigue and static testing of eyebars received from the above mentioned bridges. The service load tests were performed using loaded county gravel trucks (approximately H 15) to apply the loads to the bridges.

Strain readings were taken to determine the forces in members of the trusses. Also, deflection readings were taken of the trusses in one span and of the deck and strain readings in the floorbeams to determine the moments.

The experimental forces in the members of the truss agreed with the forces found theoretically using a determinate analysis. There were some discrepancies but these were mainly due to problems in the instrumentation. The experimental deflections of the trusses in one span were found to be much smaller than the theoretical deflections.

This was due to the partial continuity of the deck which was not taken into account in the theoretical analysis, and also due to the partial rigidity of the joints.

Deck deflections were measured at the middle of the panels with the truck on the centerline of the bridge and on the edges of the bridge. The experimental deflections were between the theoretical values for stringers assuming fixed ends and assuming pinned ends. The behavior of the deck compared quite well with that predicted by the AASHTO Specifications[4]. The current load distribution criteria, assuming S/4 as the distribution factor, indicates that for the Hubby Bridge each stringer should be designed for about 14 percent of the total weight of the truck (28 percent of a wheel load, front and rear). The test results indicated a distribution value of 10 percent to each stringer for both the centered load and the eccentric load. For the Chestnut Ford Bridges, however, the current load distribution criteria, assuming S/4.5 as the distribution factor, indicates that each stringer should be designed for 14 percent of the total weight of the truck. The test results indicate a value of 14 percent for the centered load and 19 percent for the eccentric load.

Moment cross sections for the floorbeams were found experimentally with the rear axle of the truck located over the floorbeams. The experimental results were between the theoretical values for a floorbeam assumed fixed at the ends and assumed pinned at the ends. Floorbeams 3 and 5 from the Hubby Bridge tended to behave more closely to the pinned end assumption while floorbeams 4 and 6 tended to agree more closely with the fixed end assumption.

In the fatigue tests of the tension eyebars it was found that the eye of the bar tended to be more susceptable to fatigue failure than the forgings at the intersection between the eye and the bar. Twenty-one of the 23 undamaged bars fractured in the eye while the remaining two eyebars fractured in forgings, where large initial cracks were present. Of the nine eyebars that were damaged and repaired and then tested in fatigue, only one eyebar failed in the first repair and it was a repair that had been made in the field over 40 years ago.

In the static tests different types of eyebars were found to fail in different fashions but consistent for the particular type. All of the rectangular eyebars fractured in the forgings while all of the round eyebars fractured in the bars away from the forgings. The square bars fractured both in the forgings as well as in the bars. The minimum percentage of effectiveness found in the tests was 59 percent. This compares with the 40 percent effective rule usually assumed for rating of eyebars as commonly used in Iowa.

The fatigue strength of the eyebars varied over a wide range, but it was seen that for a stress range of 14 ksi, the fatigue life of the bars was approaching 2,000,000 cycles. In the Hubby Bridge, an H 10.7 truck, in the eccentric position with an included impact factor of 30 percent will produce a live load stress range in a hanger of 14 ksi stress range. For the Chestnut Ford Bridge an H 18.3 will produce the same stress range. Assuming 10 loaded trucks of this type a day, every day of the year, it would take 28 years to reach 100,000 cycles.

It can be seen from this that the weight of this type of truck is substantially more than that usually carried by the bridges and thus, it would not be expected that there would be any reduction in the fatigue life of the members. This was observed in the overall results of the fatigue study.

CHAPTER 7. CONCLUSIONS

As a result of the tests performed on these two bridges, the following conclusions were reached.

Ultimate Load Tests

1. The behavior of the timber deck was linear up to about one-half of the ultimate load for each deck test.
2. For deck test 1 (centered load) the design percentage of the total load distributed to the most heavily loaded stringer, based on the AASHTO Specifications, is greater than the experimental percentage of the load distributed to the most heavily loaded stringer based on the deck deflection at all load levels for which this is valid.
3. The theoretical capacity of the deck for deck test 1 is approximately equal to the experimentally determined capacity of the deck.
4. For deck test 2 (eccentric load) the design percentage of the total load distributed to the most heavily loaded stringers, based on the AASHTO Specifications, is equal to or less than the experimental percentage of the load distributed to the most heavily loaded stringers based on the deck deflection at all load levels for which this is valid.
5. The theoretical capacity of the deck for deck test 2 is approximately equal to the experimentally determined capacity of the deck.
6. The deflections of the timber deck for both tests generally lie within the theoretical bounds.
7. The experimentally determined forces for the truss members agree closely with the forces for the same members from analysis. This indicates that the assumption of pinned end members is valid for this particular truss.

8. The theoretical capacity of the hangers at L_5 agrees quite closely with the load that actually caused the fracture of one of these hangers.

9. The current practice of assuming the "lap" of an eye-bar to be only 40 percent effective is quite conservative. (Additional tests are required before any recommendation on changing this assumption is warranted).

10. The natural dapping of the stringers provides sufficient lateral support of the floorbeam up to approximately 60 percent of the ultimate load.

11. The theoretical capacity of each floorbeam was approximately equal to the actual capacity of each floorbeam.

12. The ratings of the bridge and its components average about 25 percent of capacity. The ratings were fairly consistent except for the floorbeams, where the assumption on lateral support conditions for the compression flange caused considerable variation

Service Load Tests

1. Fatigue fractures tend to be governed by the characteristics of the eye while the static fractures tend to be governed by the quality of the forgings.

2. The fatigue life of the eyebars after being damaged and repaired was not appreciably different from that of an undamaged eyebar.

3. The experimentally determined forces of the truss members for both the Hubby Bridge and Chestnut Ford Bridge agree closely with the forces found from the theoretical analysis assuming pinned connections, this indicates that the assumption of pinned end members is valid for these particular trusses.

4. Since the truck used for the experimental loading was approximately an H 15 and all ratings for the critical bridge

components provided by the cooperating agencies[2] were less than H 15 (ranged from H 2 - H 13), the results show that determinate methods are valid for analyzing the bridge for loads in the range of rating levels.

5. The current practice of assuming the "lap", or forging, in an eyebar to be only 40 percent effective is conservative. The minimum found during testing was 59 percent.

6. The current AASHTO Load Distribution criteria for the Hubby Bridge of S/4 is more than adequate (S/4.5 could be used if it is considered to be a multiple layer bridge deck.)

7. The current AASHTO Load Distribution criteria for the Chestnut Ford Bridge deck of S/4.5 (strip type deck) agrees very closely to the centrally loaded truck but does not agree with the truck when in the eccentric position. This may be misleading in that the maximum deflection measured in each case was essentially the same (i.e., the same maximum moment).

CHAPTER 8. PROJECT FINDINGS

On the basis of the research program conducted, the following findings can be stated. These findings are, of course, based on limited tests and may be subject to modifications as additional information becomes available. Although these findings are developed for this type of bridge or components, many of the findings can be applied judiciously to bridges of a similar nature.

1. The ultimate strength of a bridge truss, as determined by using conventional determinate analysis of the truss, is a reasonable estimate of the actual ultimate strength of the truss.

2. The ultimate strength of the bridge (including its components) is substantially higher in all cases than that found using present rating criteria (operating level).

3. Because of the methods used in the fabrication and design, a natural indeterminancy is built into the system. Thus, unless a key member of the bridge is ruptured, such as an upper or lower chord, the bridge will not collapse catastrophically and will be able to sustain a reasonable load.

4. Gross deflections of the trusses and the floor system will occur prior to failure.

5. The live load stresses in the truss member are generally slightly lower than those predicted using a conventional determinate analysis.

6. The remaining fatigue life of the tension members of the truss is such that with reasonable levels of live load, bridge life should be determined by its serviceability and degree of deterioration and not by fatigue considerations.

7. If a failure in a member of the truss should occur due to an accident, etc., repair procedures such as those used in the supplementary tests for fatigue and static strength will

result in the structure having essentially the same load capacity in static as well as fatigue strength.

8. The static strength of the forging at the eye of the bar can be assumed to be about 75 percent of the ultimate strength of the bar cross section. The average of the static tests was higher than this value, but there was one bar lower, therefore, this value appears to be a reasonable compromise which can be used for rating purposes.

9. The only areas of the bridge which cannot be effectively inspected are the joints where the eyebars intersect. However, in item no. 6 it was stated that the load carrying capacity of the bridge should not be determined by the fatigue strength of the eyebars. Therefore, the forgings of the eye to the bar should be closely inspected to detect flaws which would be critical in the case of an occasional over load.

10. Current AASHTO load distribution criteria are adequate for the design and rating of the floor system, although sometimes conservative.

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2. Sanders, W. W., Jr., F. W. Klaiber, H. A. Elleby and L. W. Timm, "Ultimate Load Behavior of Full-Scale Highway Truss Bridges: Phase I - Ultimate Load Tests of the Hubby Bridge - Boone County." Interim report to Iowa State Highway Commission, Engineering Research Institute, Iowa State University, Ames, April 1975.

3. Sanders, W. W., Jr., H. A. Elleby, F. W. Klaiber and M. D. Reeves, "Ultimate Load Behavior of Full-Scale Highway Truss Bridges: Phase II - Service Load and Supplementary Tests." Interim report to Iowa Department of Transportation, Engineering Research Institute, Iowa State University, Ames, August 1975.

4. American Association of State Highway Officials. Standard Specifications for Highway Bridges - Eleventh Edition. American Association of State Highway Officials, Washington, D.C., 1973.

5. American Association of State Highway Officials. Manual For Maintenance Inspection of Bridges. American Association of State Highway Officials, Washington, D.C., 1970.

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The staff for the University were all from the Structural Engineering Section of the Department of Civil Engineering. The principal staff included Dr. W. W. Sanders, Jr. (Project Investigator), Dr. H. A. Elleby (Co-Investigator), and Dr. F. W. Klaiber. Assisting in the research were faculty members (Dr. M. L. Porter, Dr. L. F. Greimann and Professor D. D. Girton), graduate students (L. W. Timm, M. D. Reeves, J. P. Sorensen, L. P. Selberg, and K. A. McDowell), and undergraduate students (T. C. Wilson and J. W. Coleman).

An Advisory Committee was formed to assist and guide the research program. The committee consisted of representatives of the affected agencies and included:

W. W. Sanders, Jr., Iowa State University (Chairman)
H. A. Elleby, Iowa State University
S. E. Roberts, Iowa Department of Transportation
J. P. Harkin, Iowa Department of Transportation
E. J. O'Conner, Iowa Department of Transportation

W. D. Ashton, U.S. Army - Corps of Engineers
S. S. Bhala, Federal Highway Administration
C. F. Schnoor, Boone County (County Engineer)
G. R. Hardy, Dallas County (County Engineer)
R. E. Van Gundy, Polk County (County Engineer)

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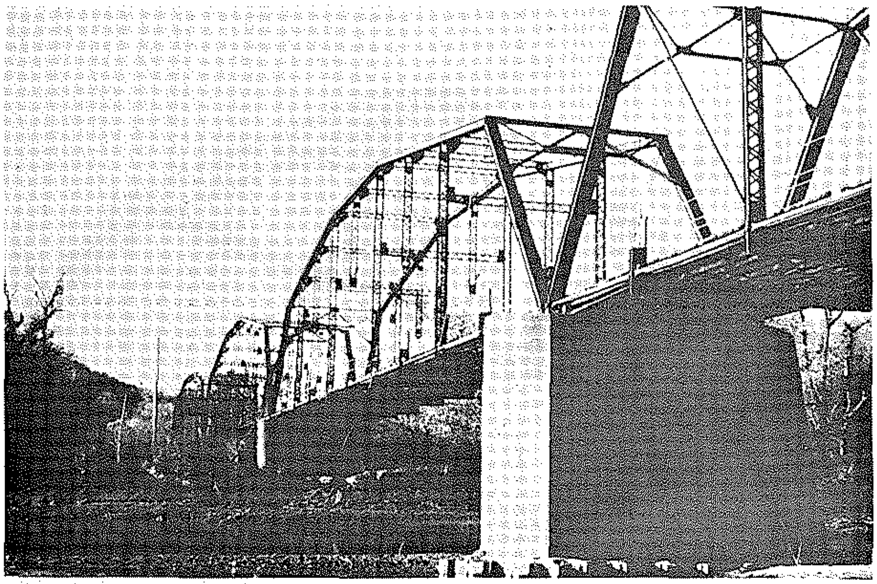
FIGURES



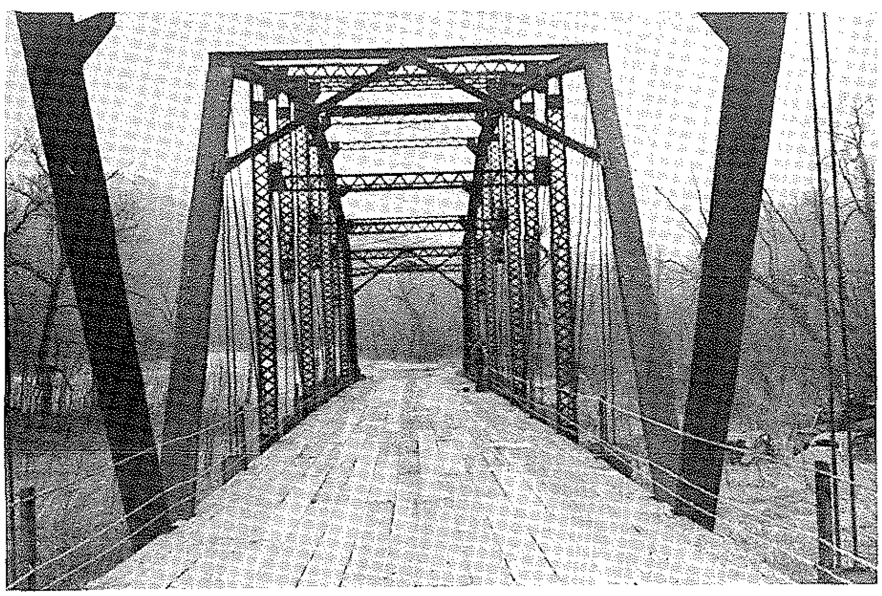
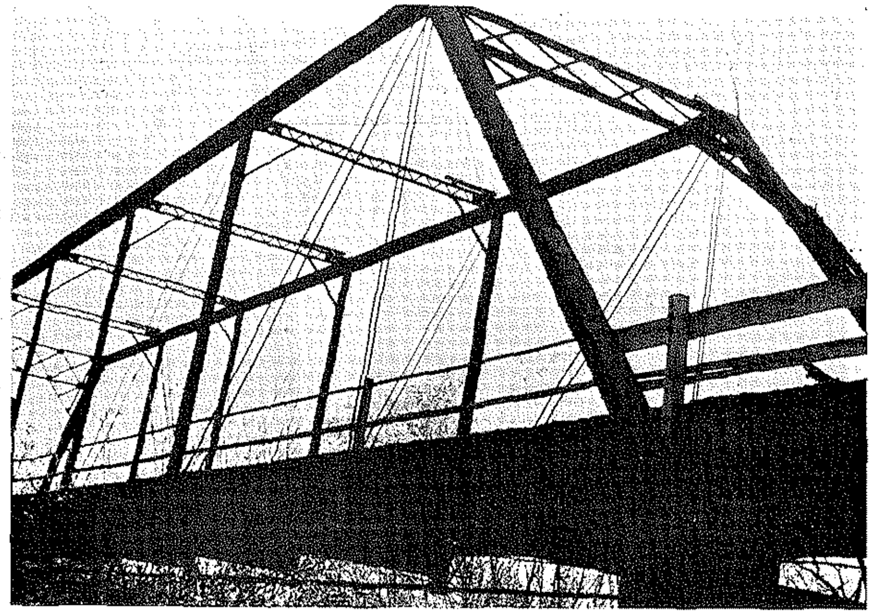


Fig. 1a. Photographs of the Hubby Bridge.



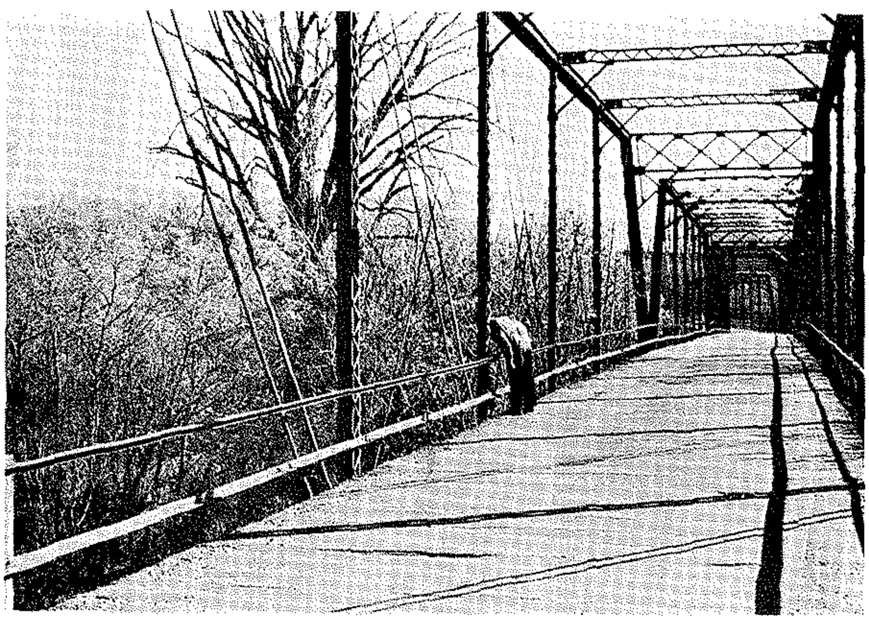
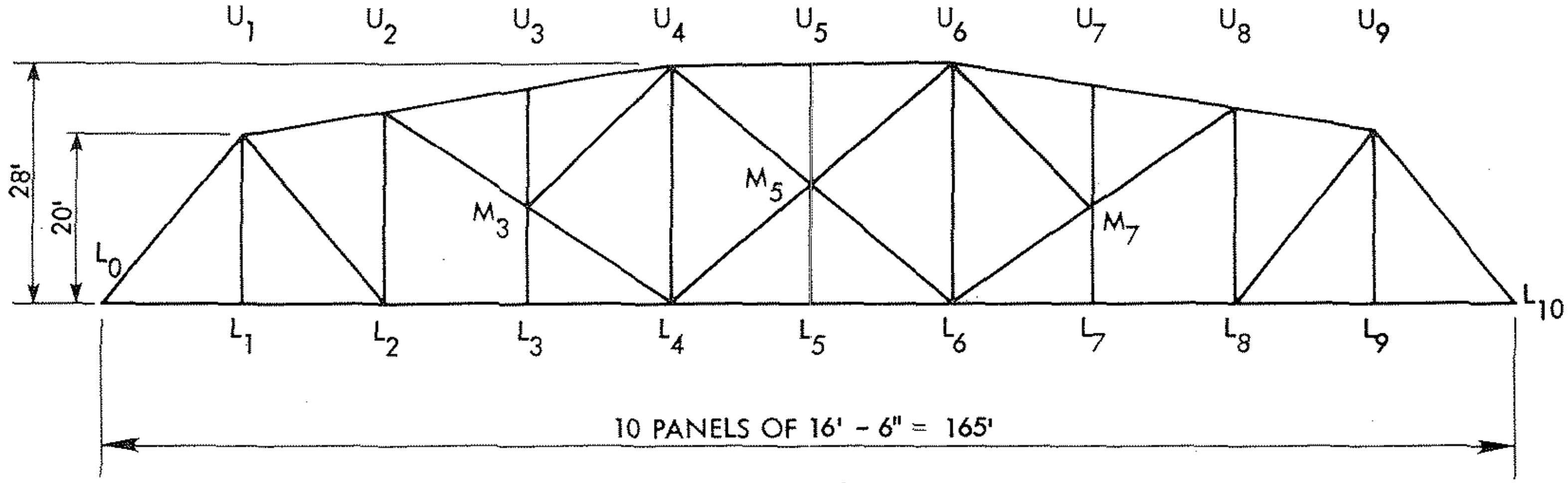
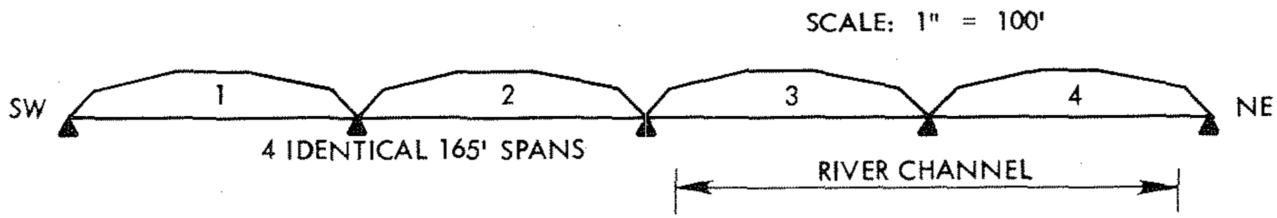


Fig. 1b. Photographs of the Chestnut Ford Bridge.

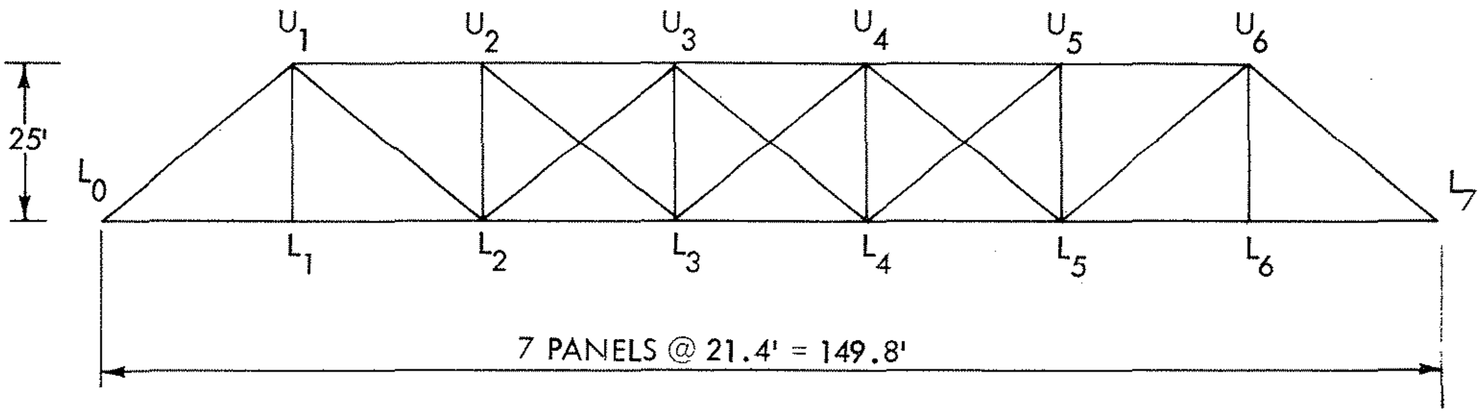


Member layout.

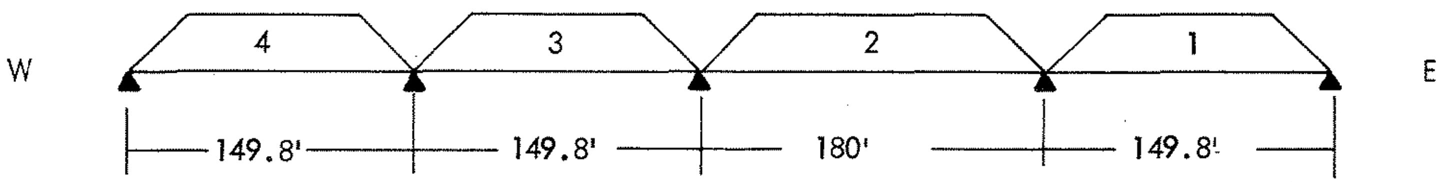


General layout.

Fig. 2a. Details of the Hubby Bridge.

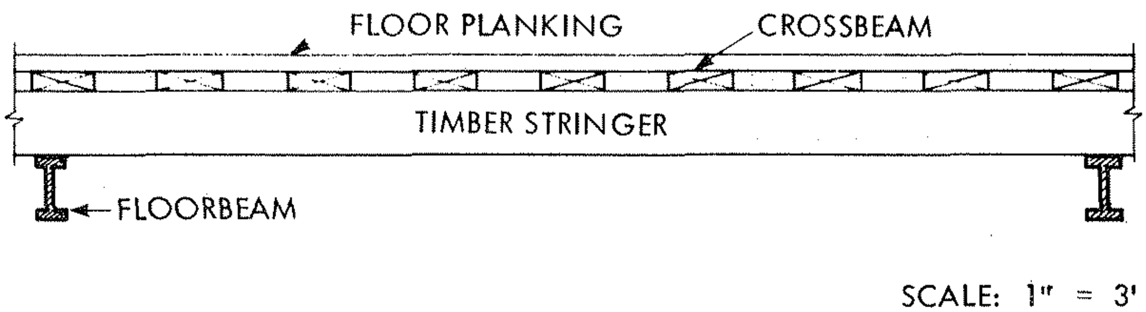


Member layout spans 1, 3, 4.

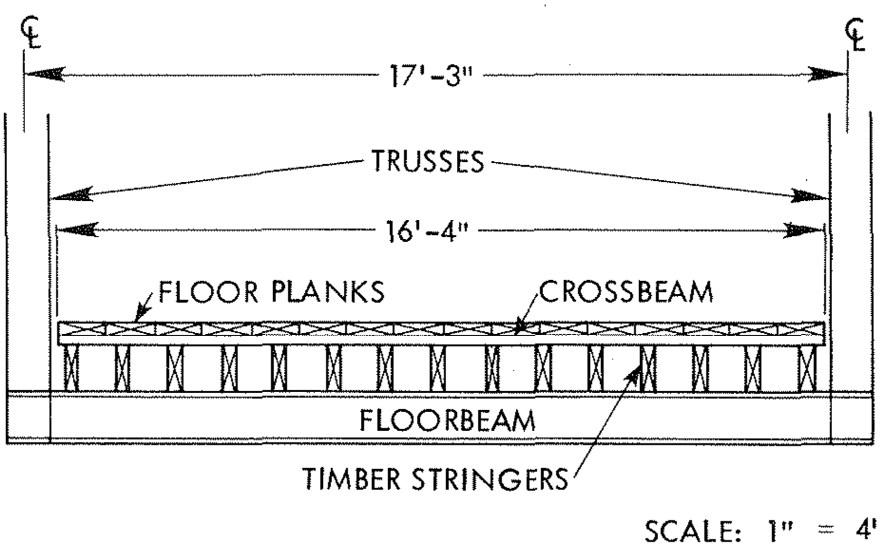


General layout.

Fig. 2b. Details of the Chestnut Ford Bridge.

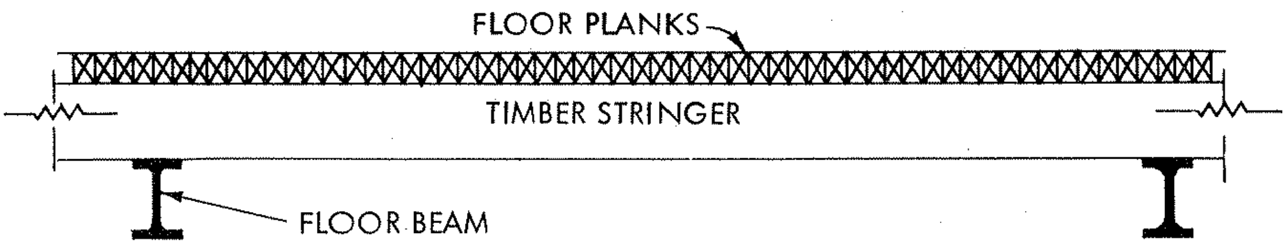


a. Elevation view.

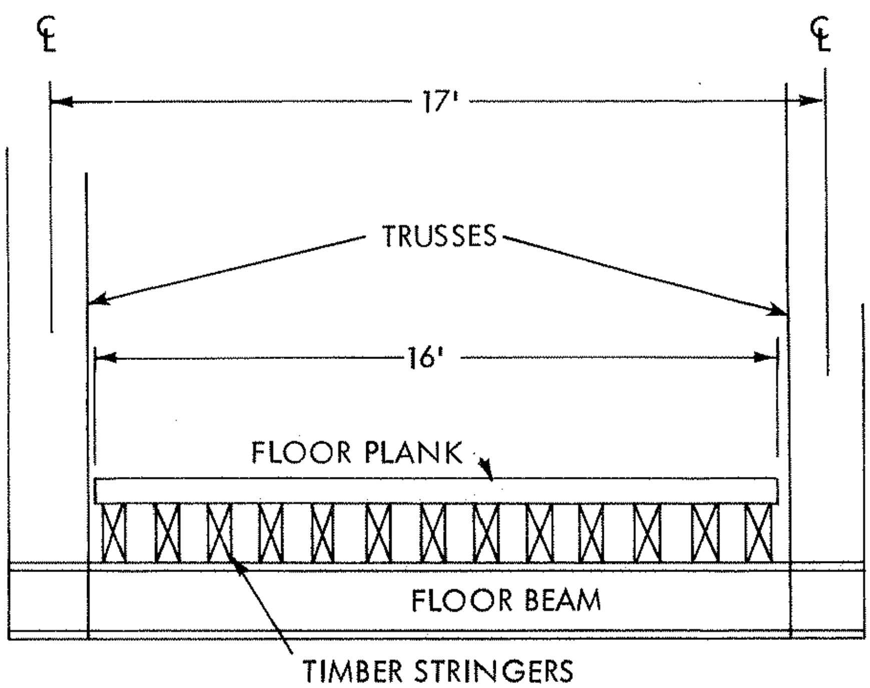


b. End view.

Fig. 3. Timber deck layout - Hubby Bridge.



a. Elevation view.



b. End view.

Fig. 4. Timber deck layout - Chestnut Ford Bridge.

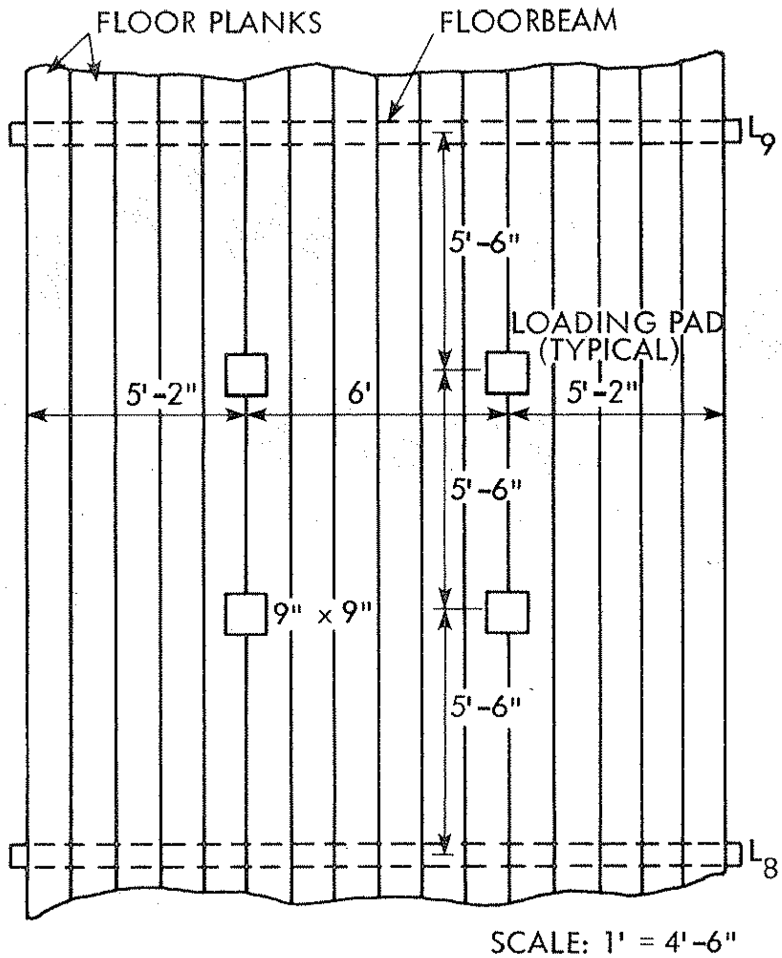


Fig. 5. Load location for deck test 1 (plan view).

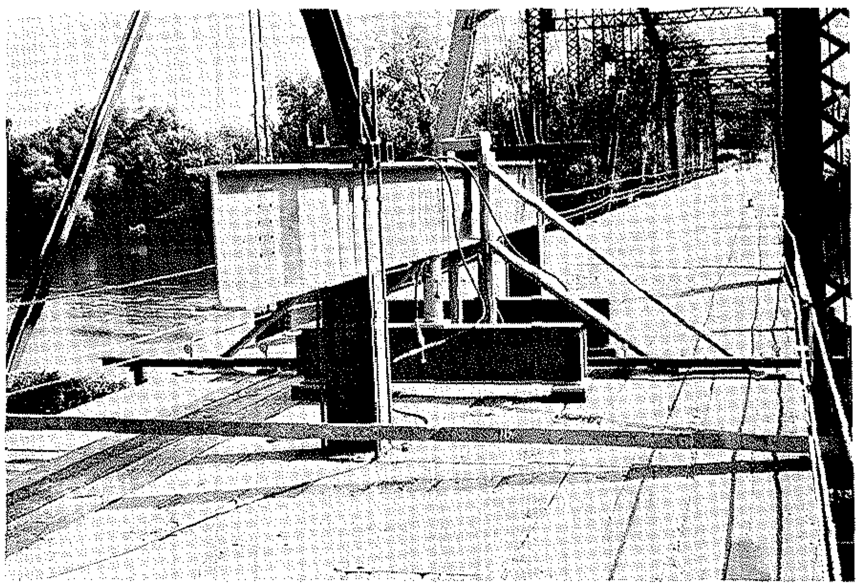


Fig. 6. Photograph of deck test 1 setup.

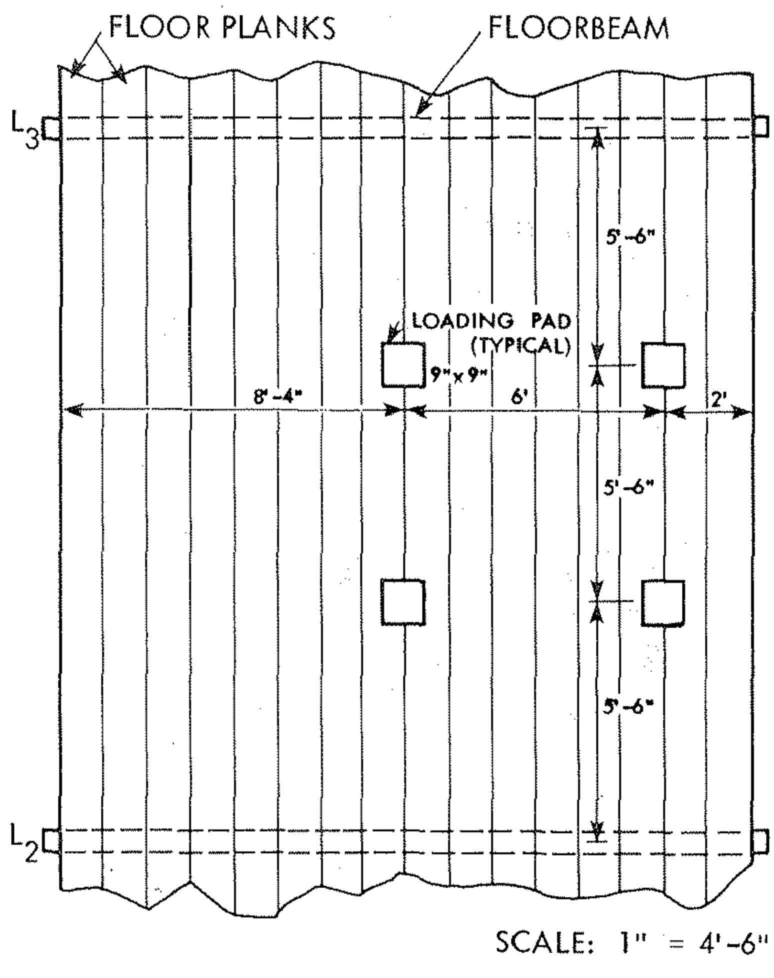


Fig. 7. Load location for deck test 2 (plan view).

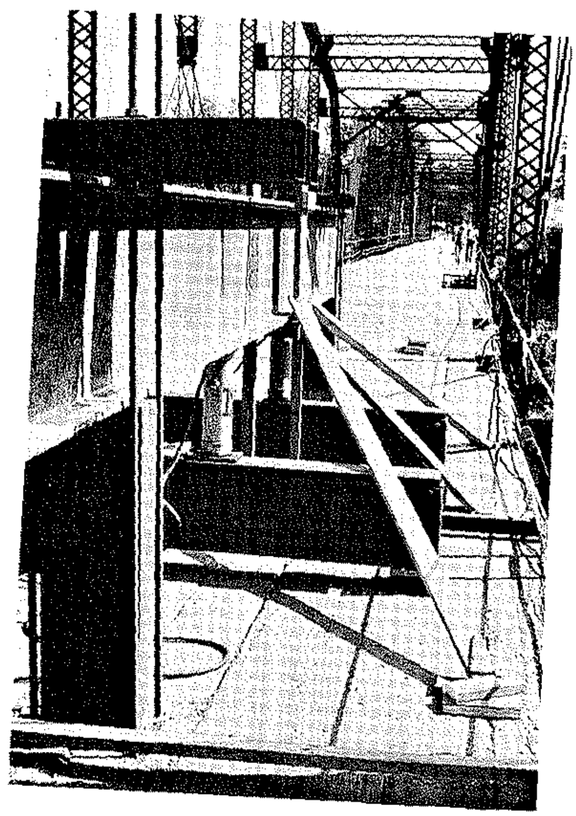


Fig. 8. Photograph of deck test 2 setup.

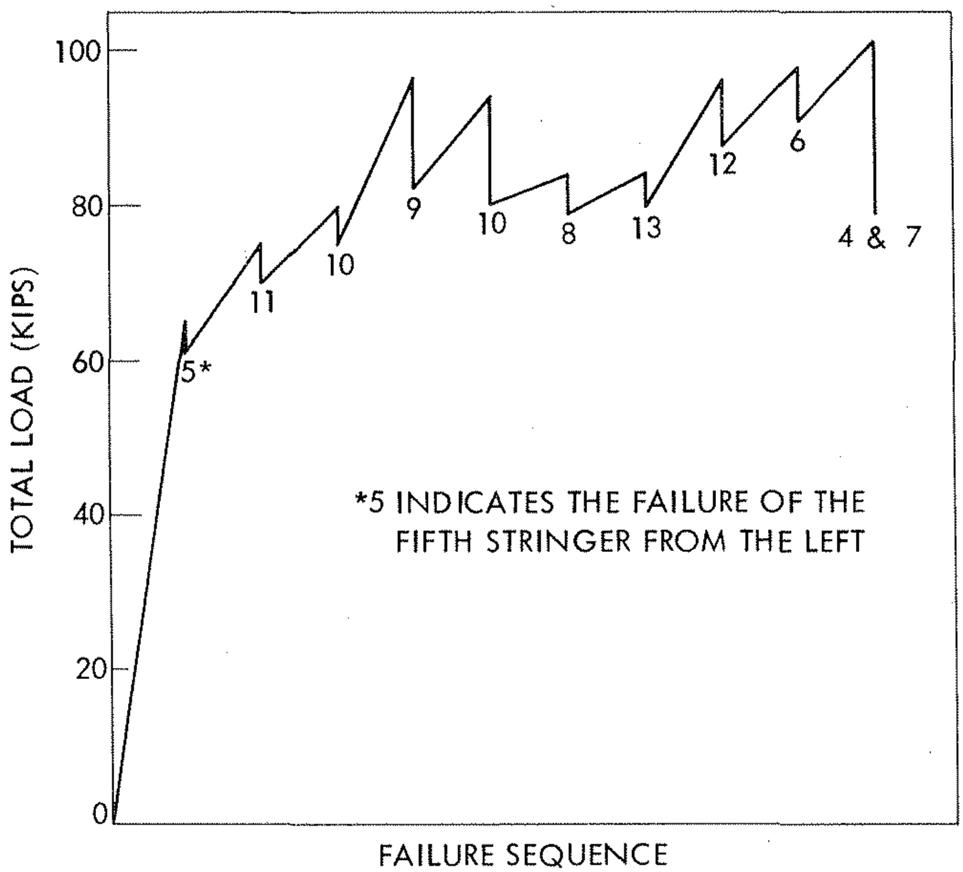


Fig. 9. Load history for deck test 1.

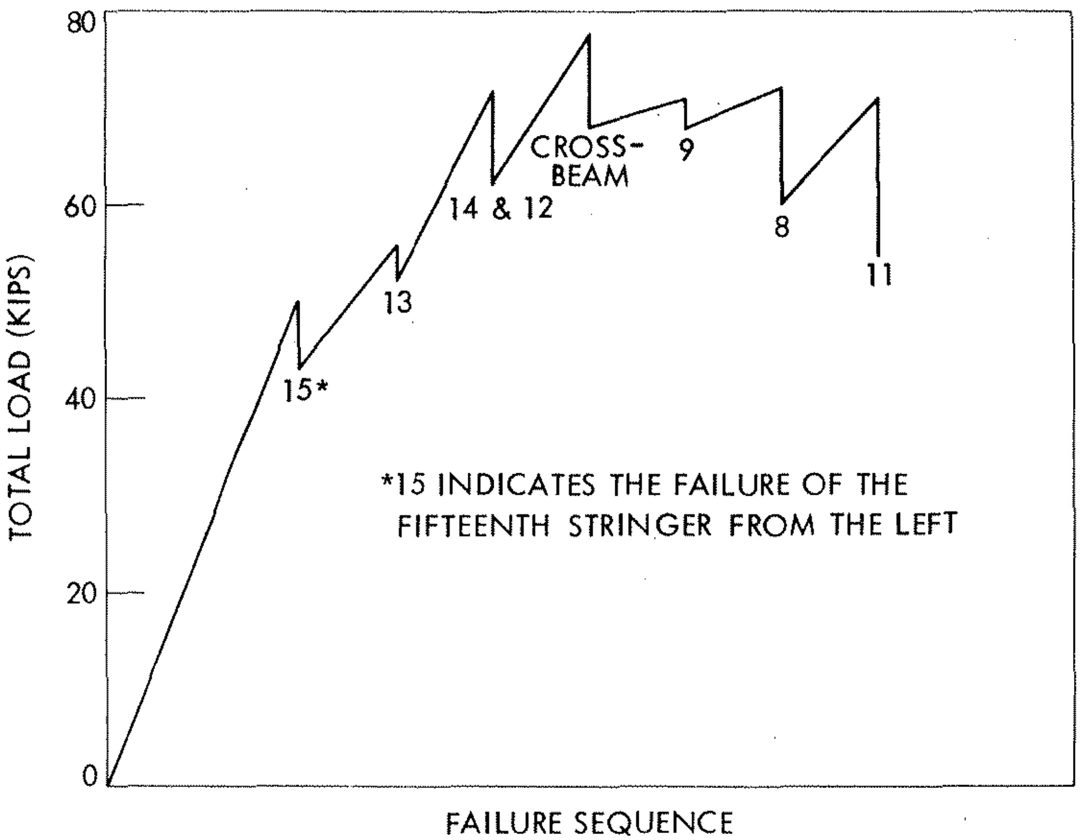


Fig. 10. Load history for deck test 2.

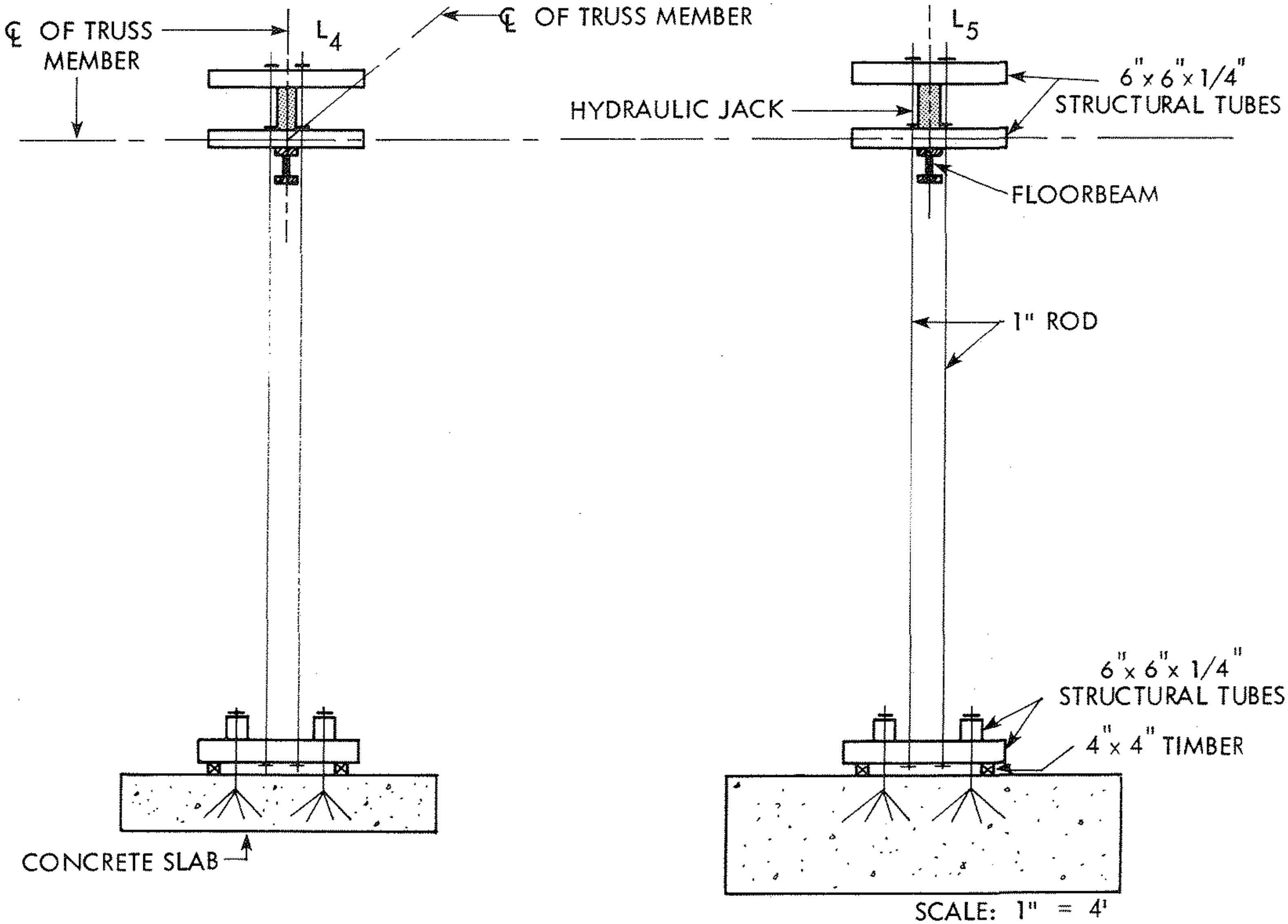


Fig. 11. Loading system details (elevation view).

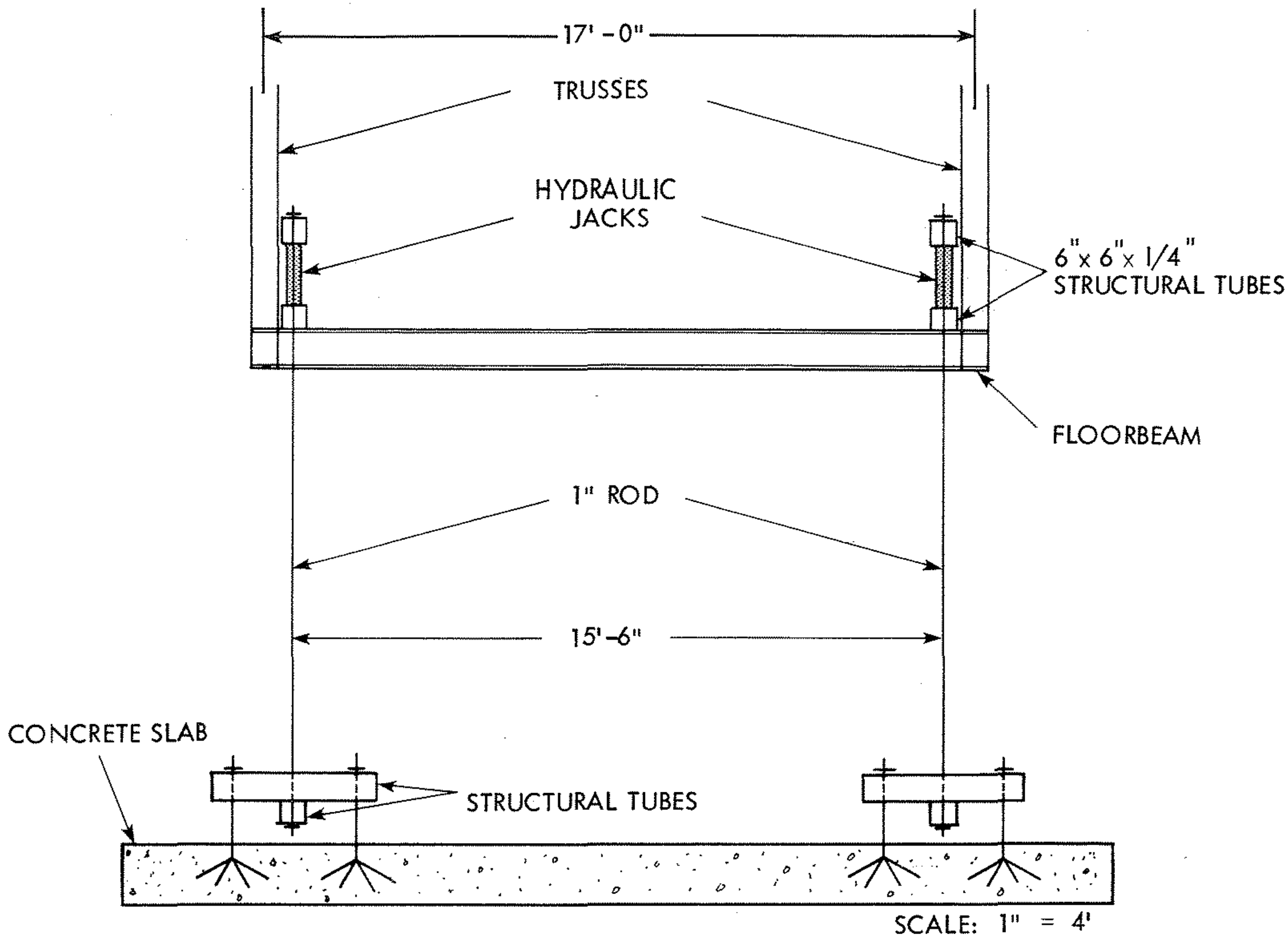


Fig. 12. Loading system details (end view).

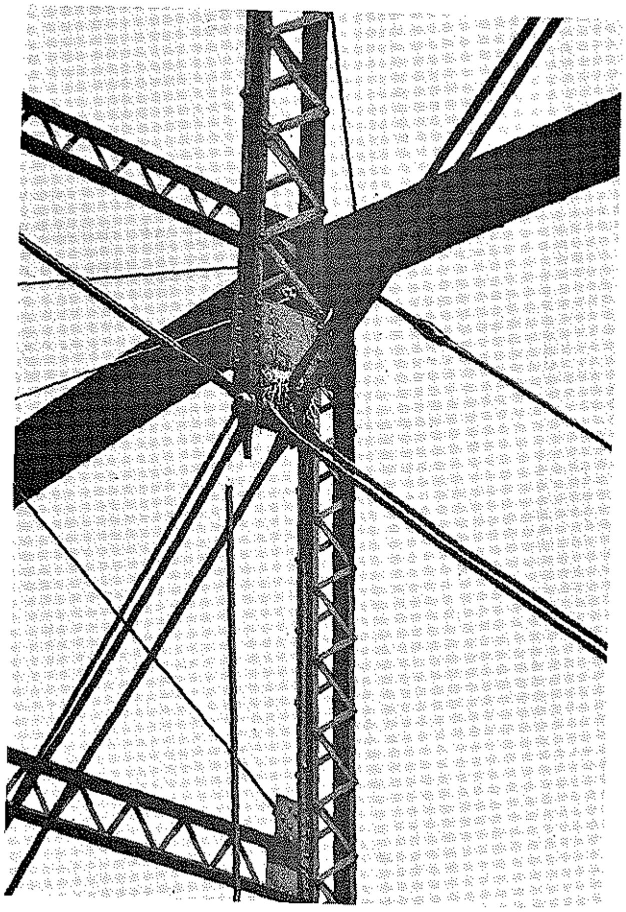


Fig. 13. Photograph showing location of failure of member L_5M_5.

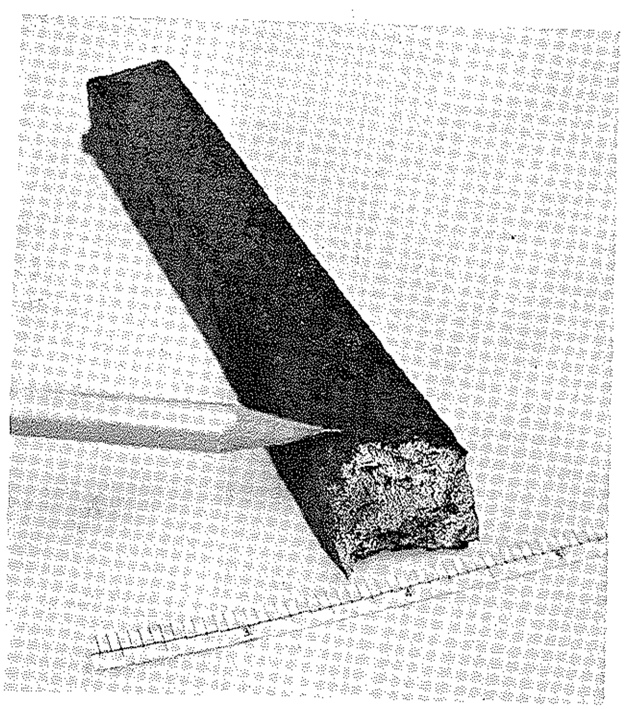


Fig. 14. Photograph of fracture.

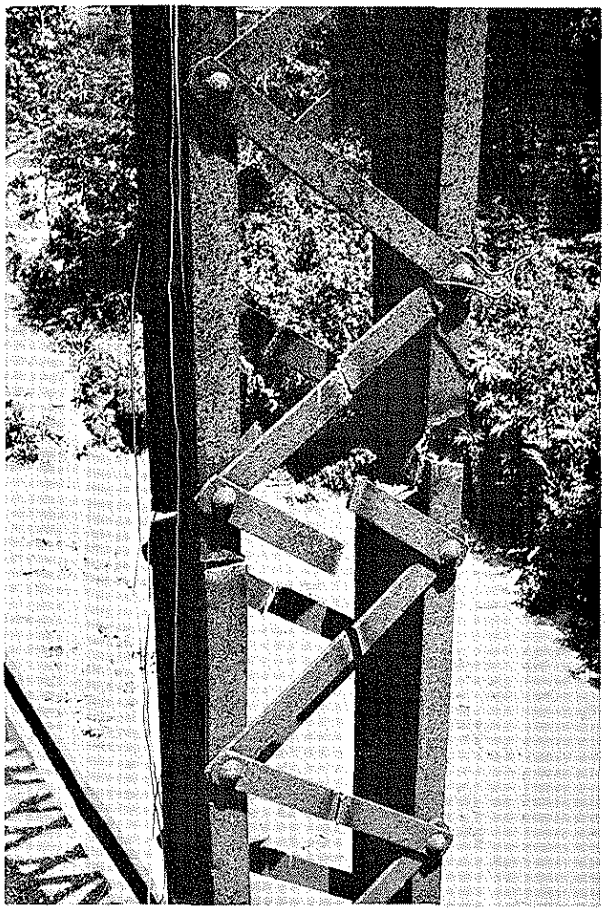


Fig. 15. Photograph of damaged member after collapsing upon itself.

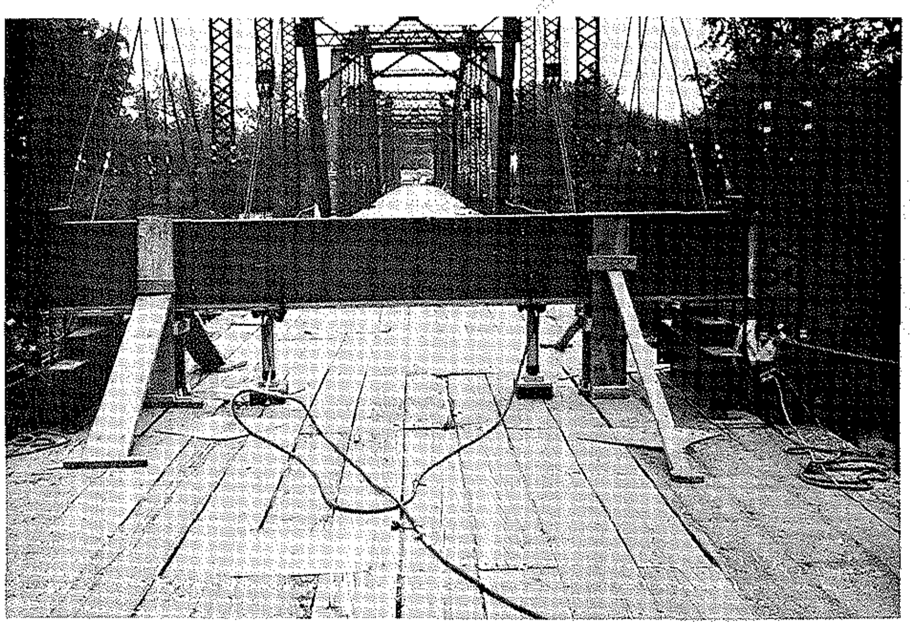


Fig. 16. Photograph of floorbeam test setup.

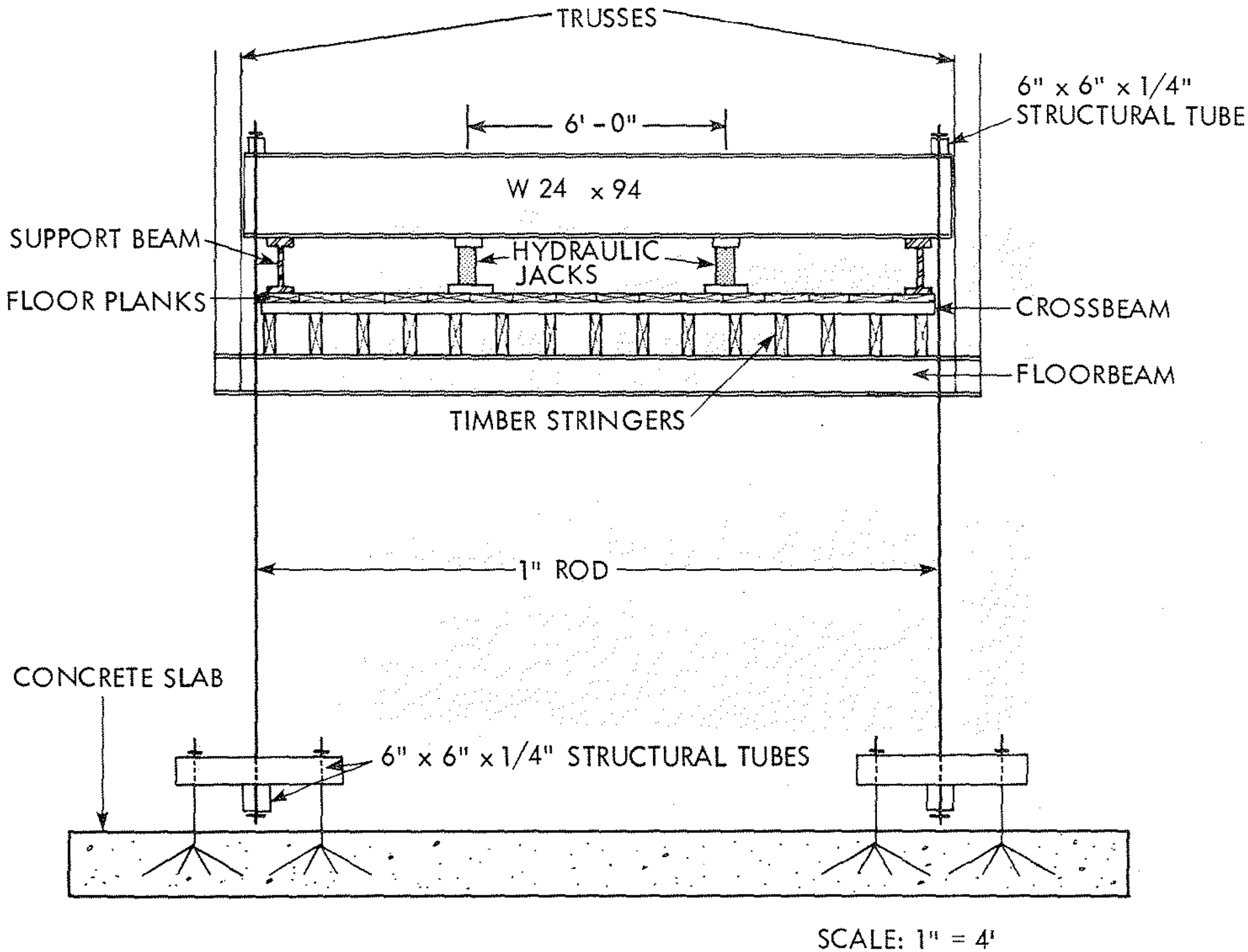


Fig. 17. Floorbeam test setup (elevation view).

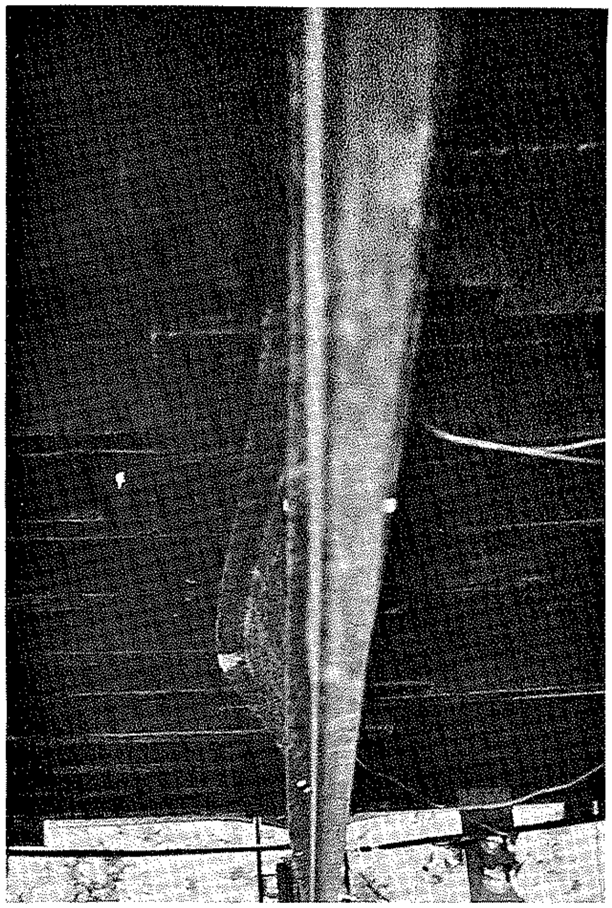


Fig. 18. Photograph of buckling of compression flange of floorbeam 5.

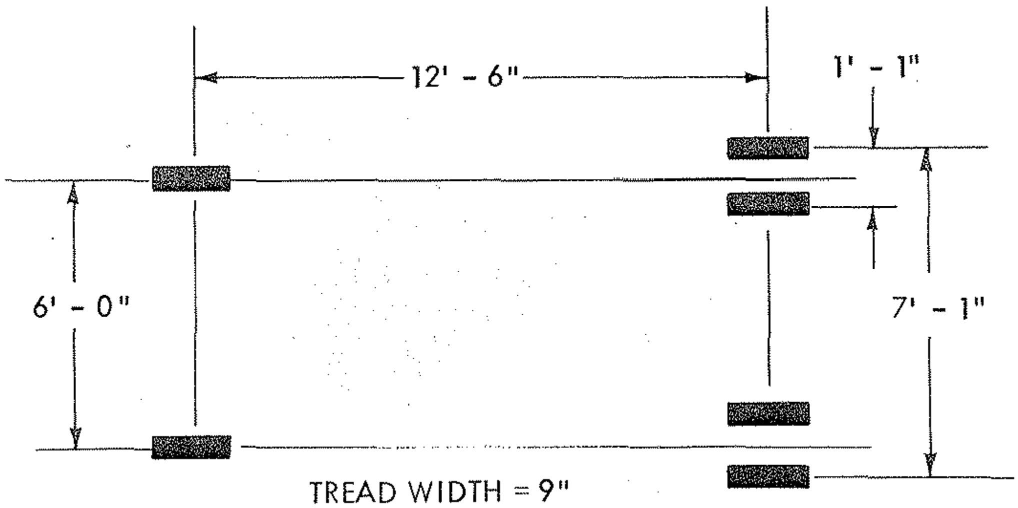


Fig. 19. Description of truck for Hubby Bridge testing.

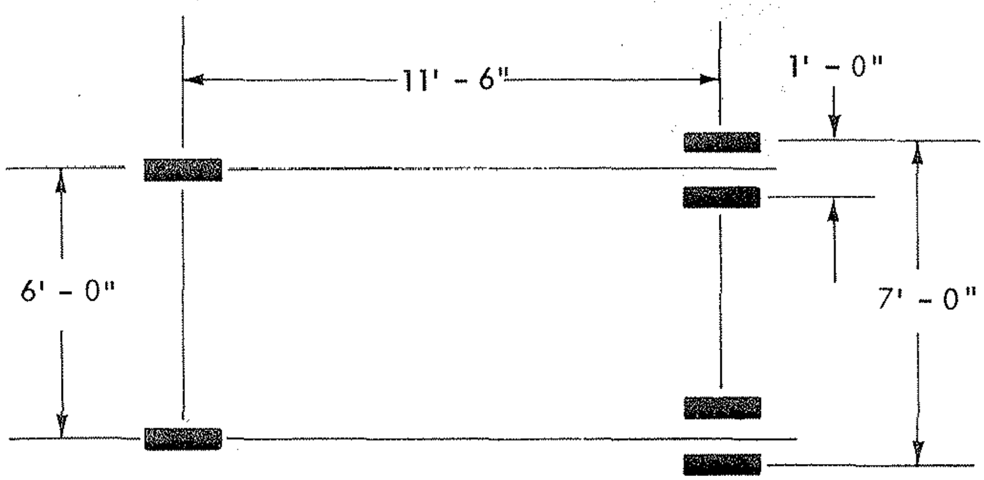


Fig. 20. Description of truck for Chestnut Ford Bridge testing.

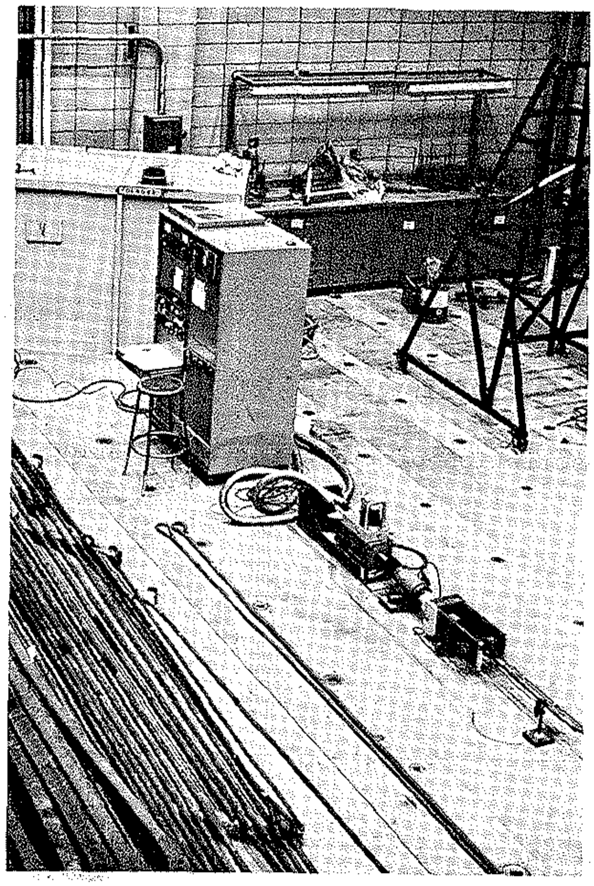
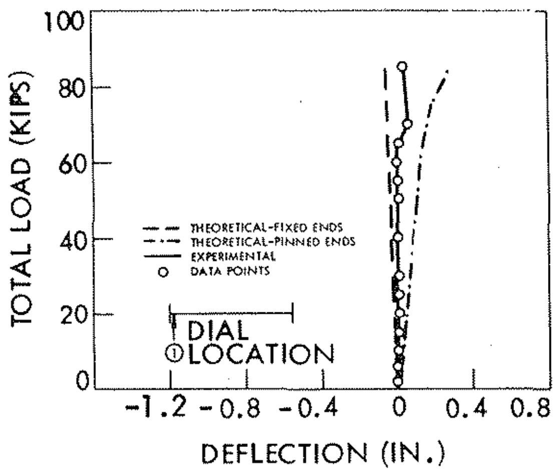
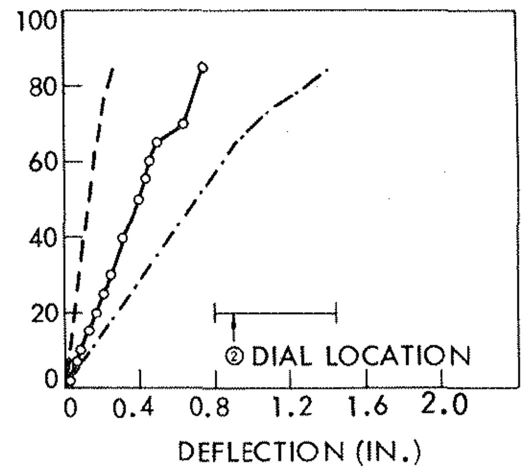


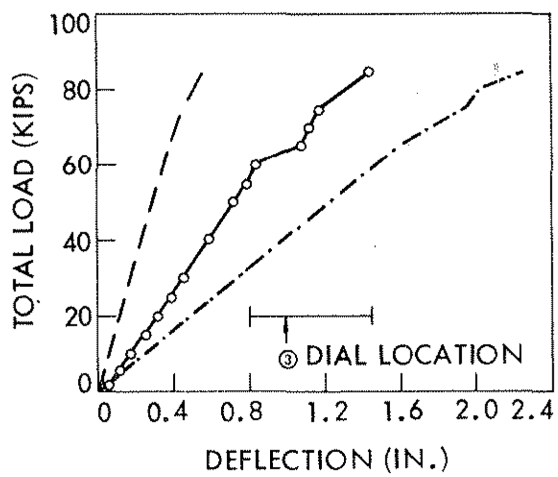
Fig. 21. Photograph of fatigue apparatus.



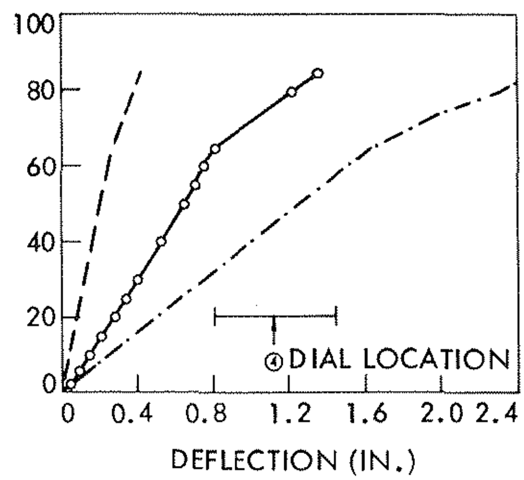
a. Position 1.



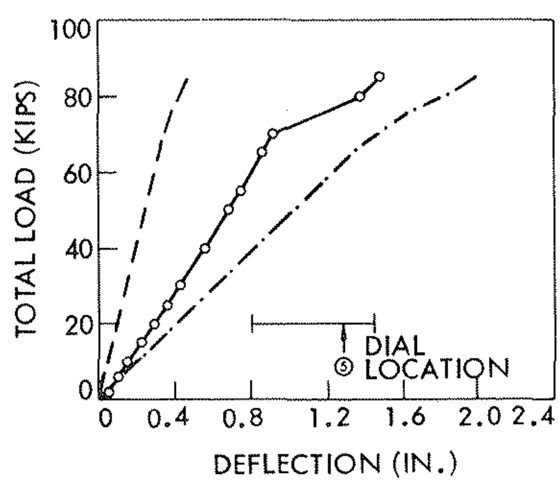
b. Position 2.



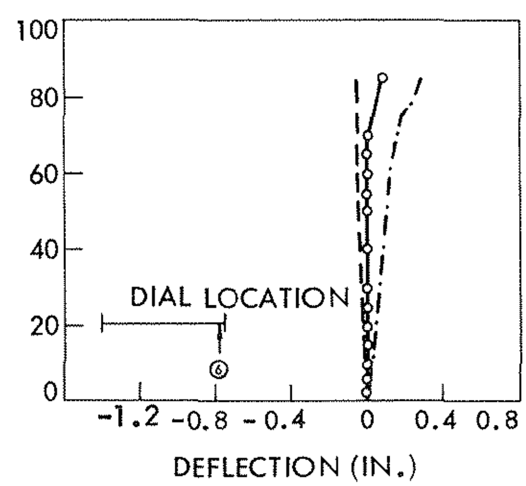
c. Position 3.



d. Position 4.

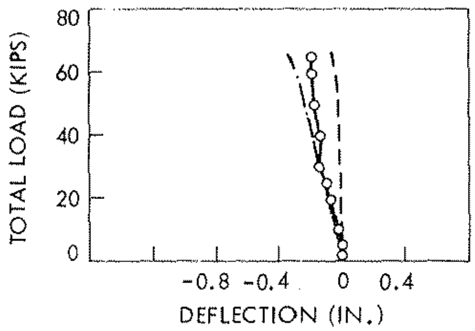


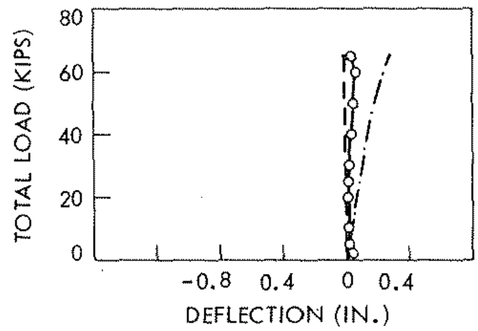
e. Position 5.

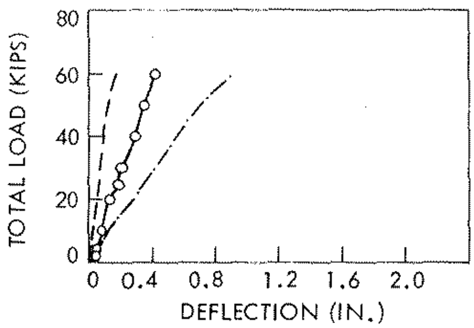


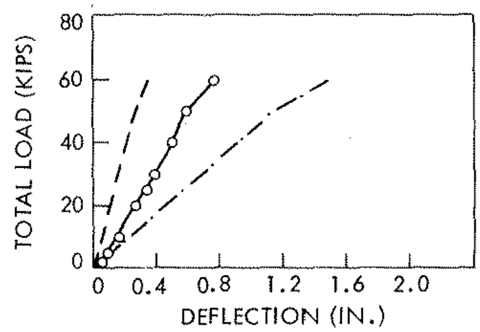
f. Position 6.

Fig. 22. Load-deflection for deck test 1.

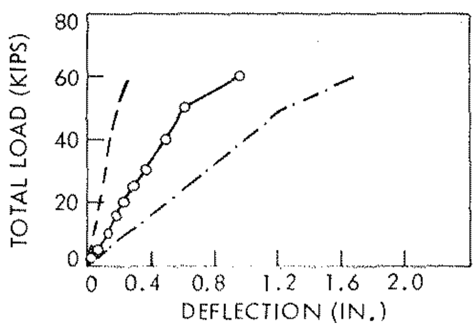




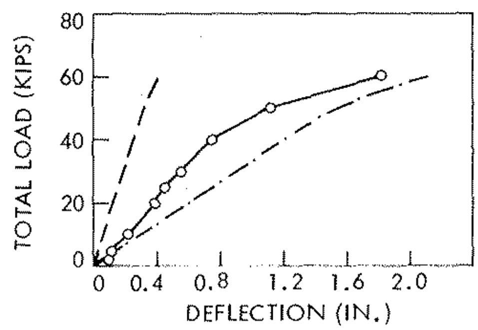




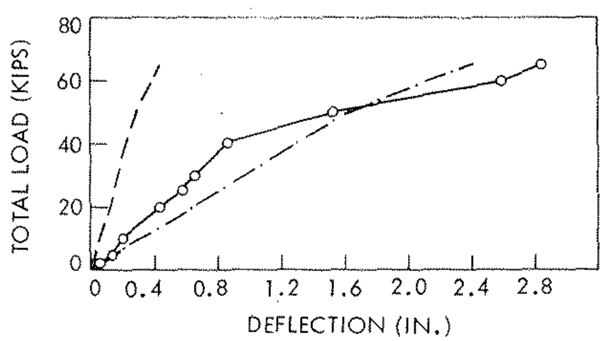
d. Position 4.



e. Position 5.



f. Position 6.



g. Position 7.

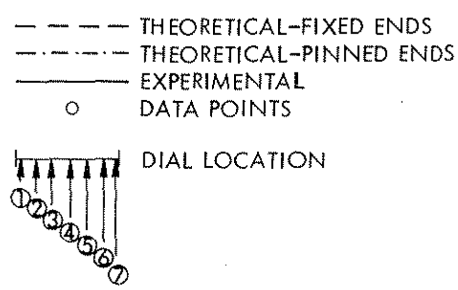


Fig. 23. Load-deflection for deck test 2.

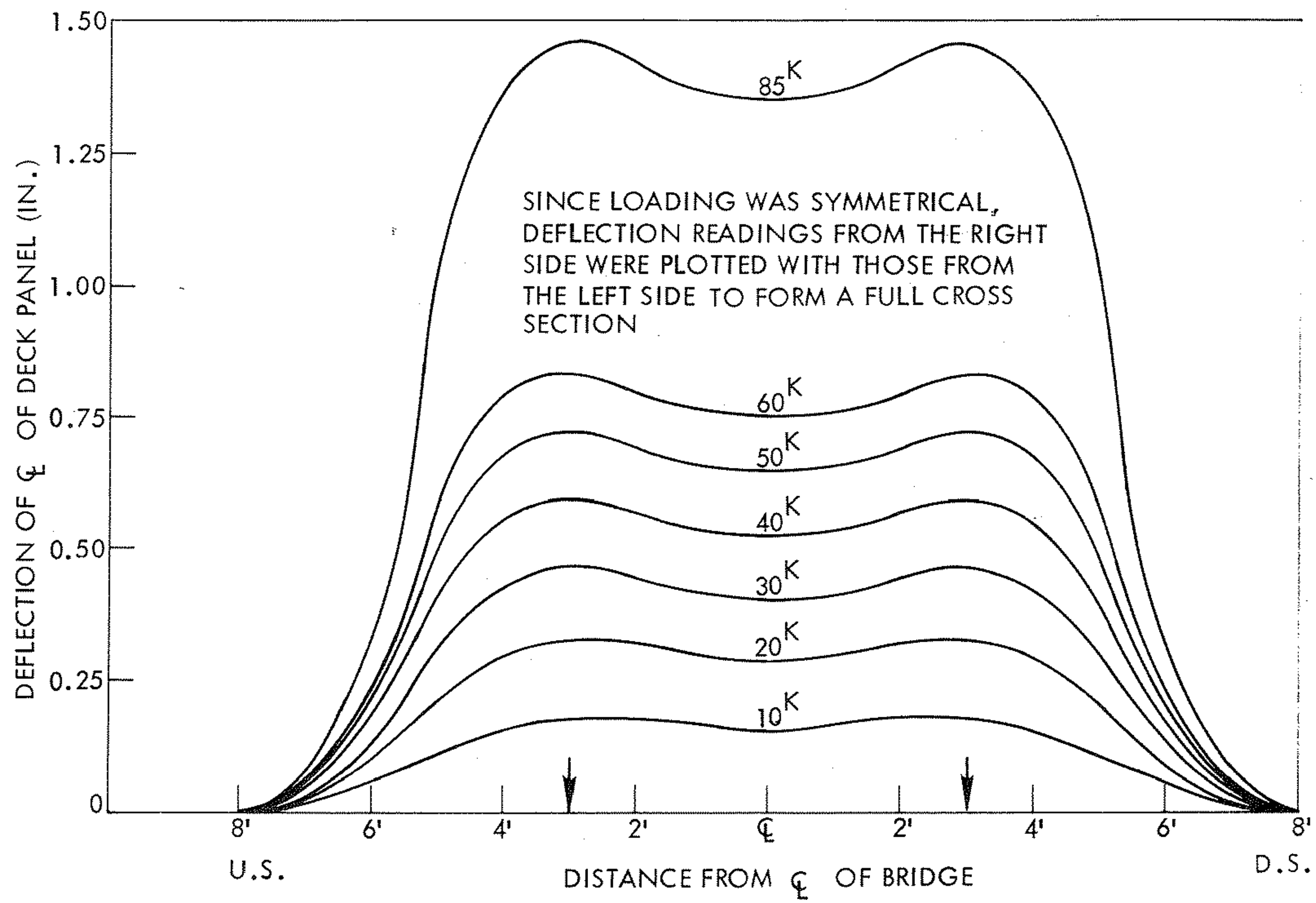


Fig. 24. Deflection cross section at mid-span of deck panel for deck test 1 at various loads.

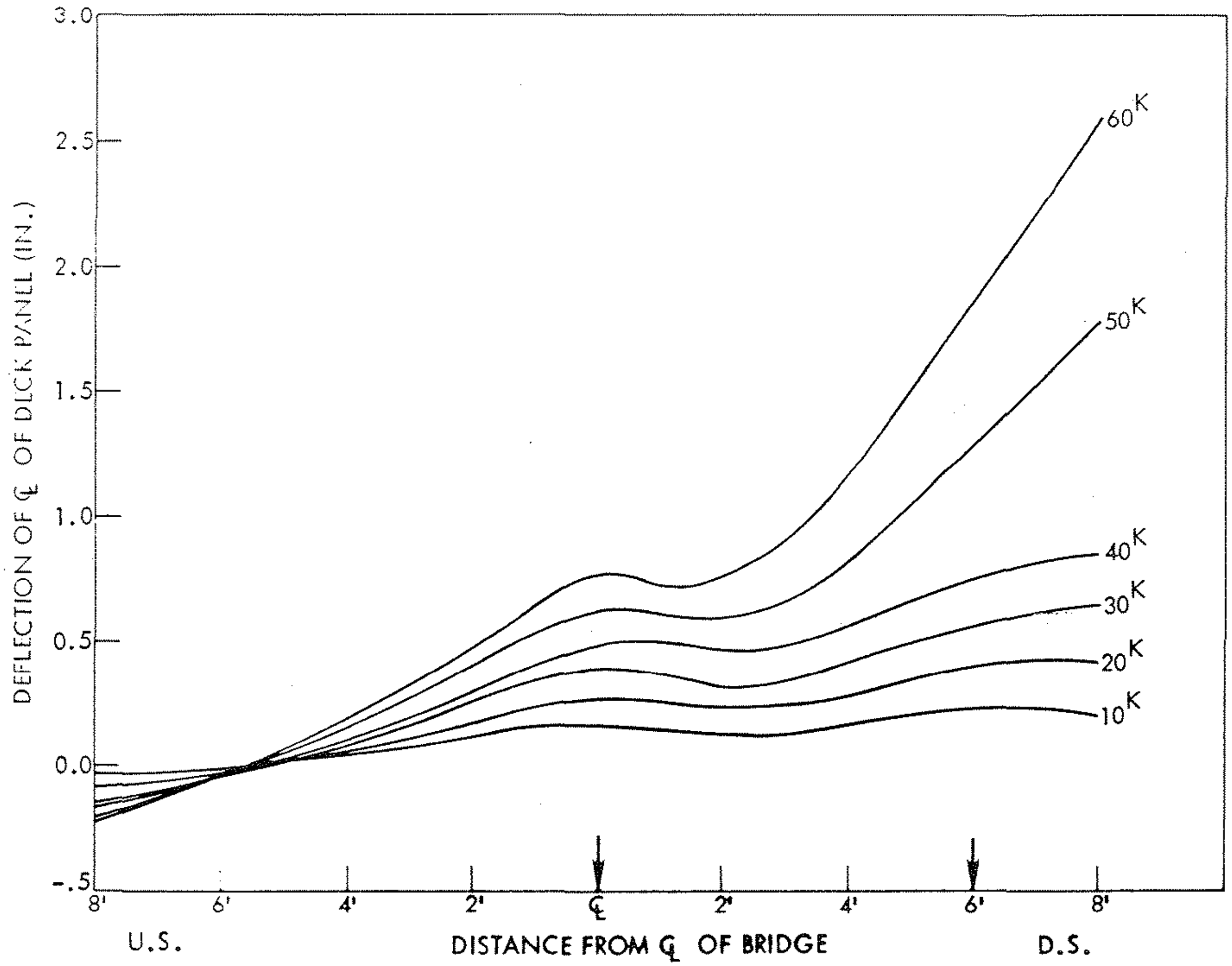


Fig. 25. Deflection cross section at mid-span of deck panel for deck test 2 at various loads.

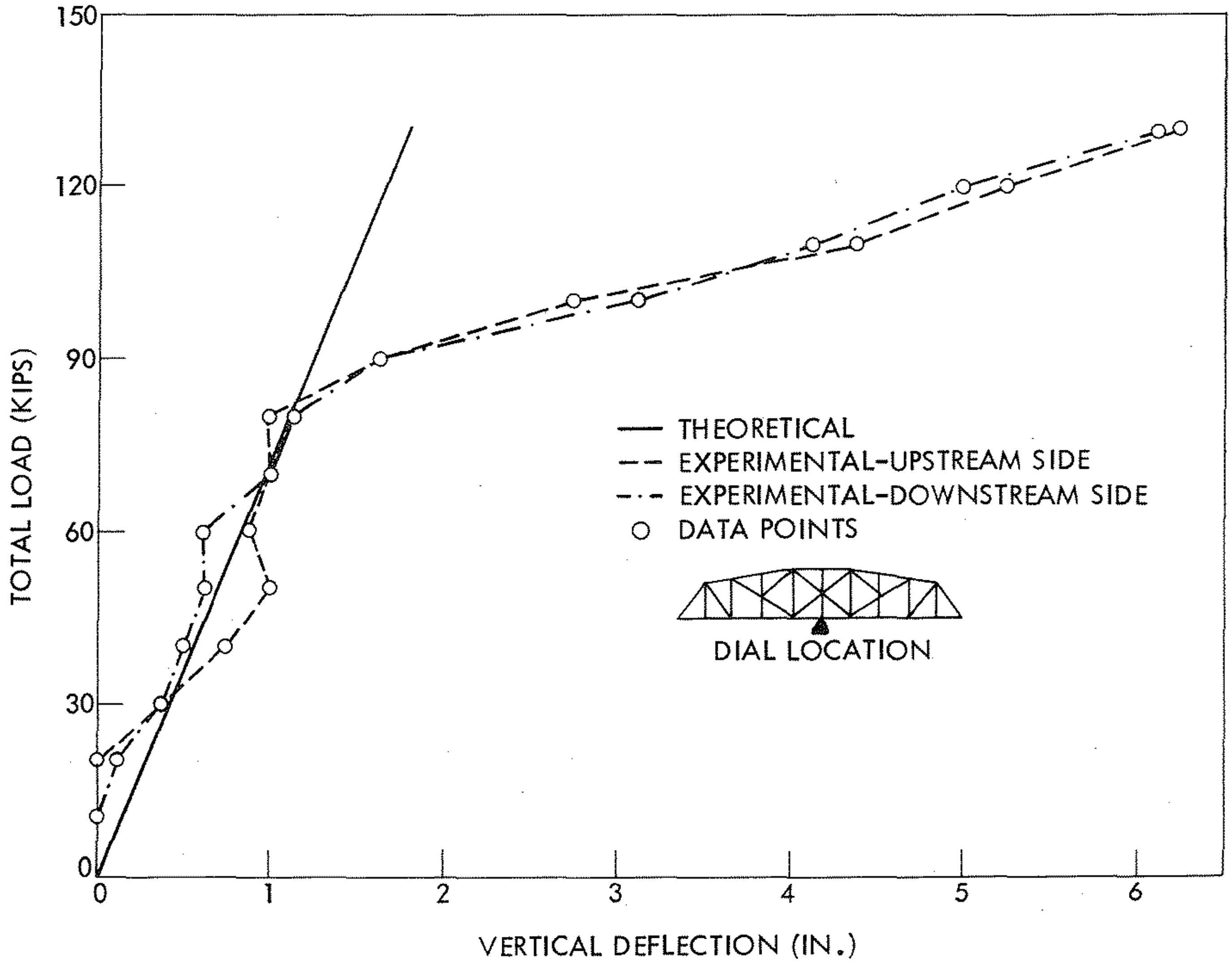


Fig. 26. Total load-vertical deflection at L_5 for truss test - Span 2, Hubby Bridge.

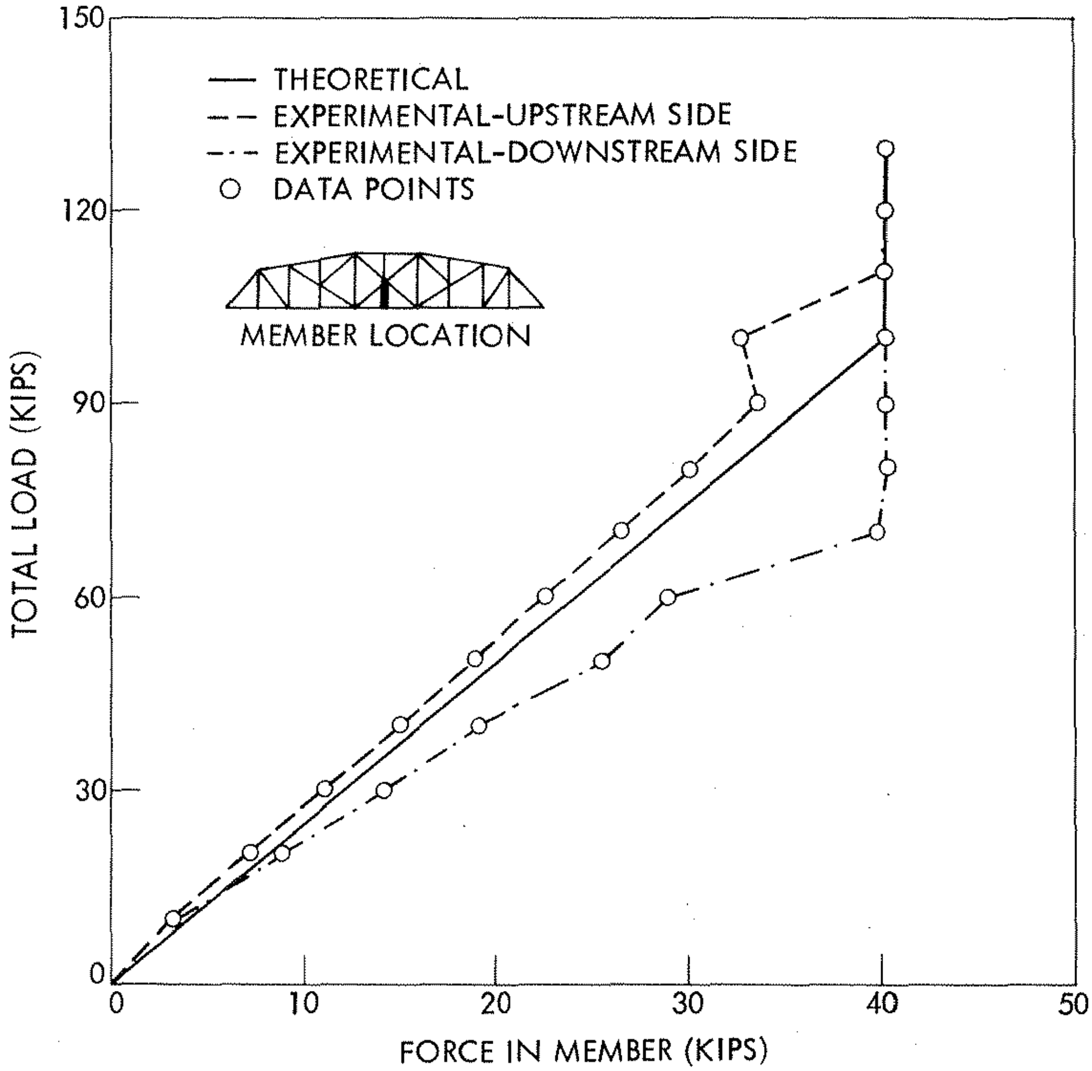


Fig. 27. Total load-force in member L_5M_5 - Span 2, Hubby Bridge.

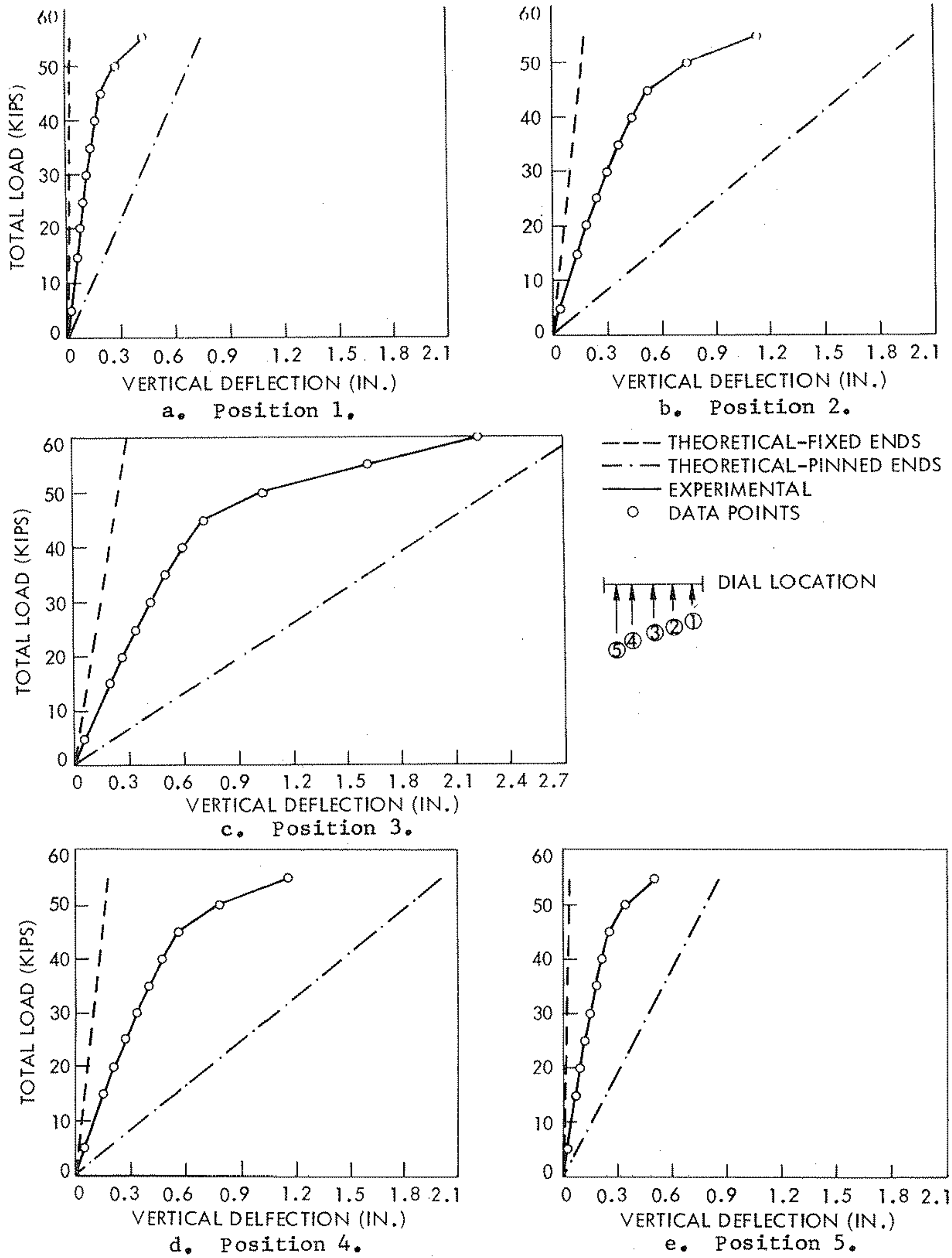


Fig. 28. Load-deflection for floorbeam test at L_4.

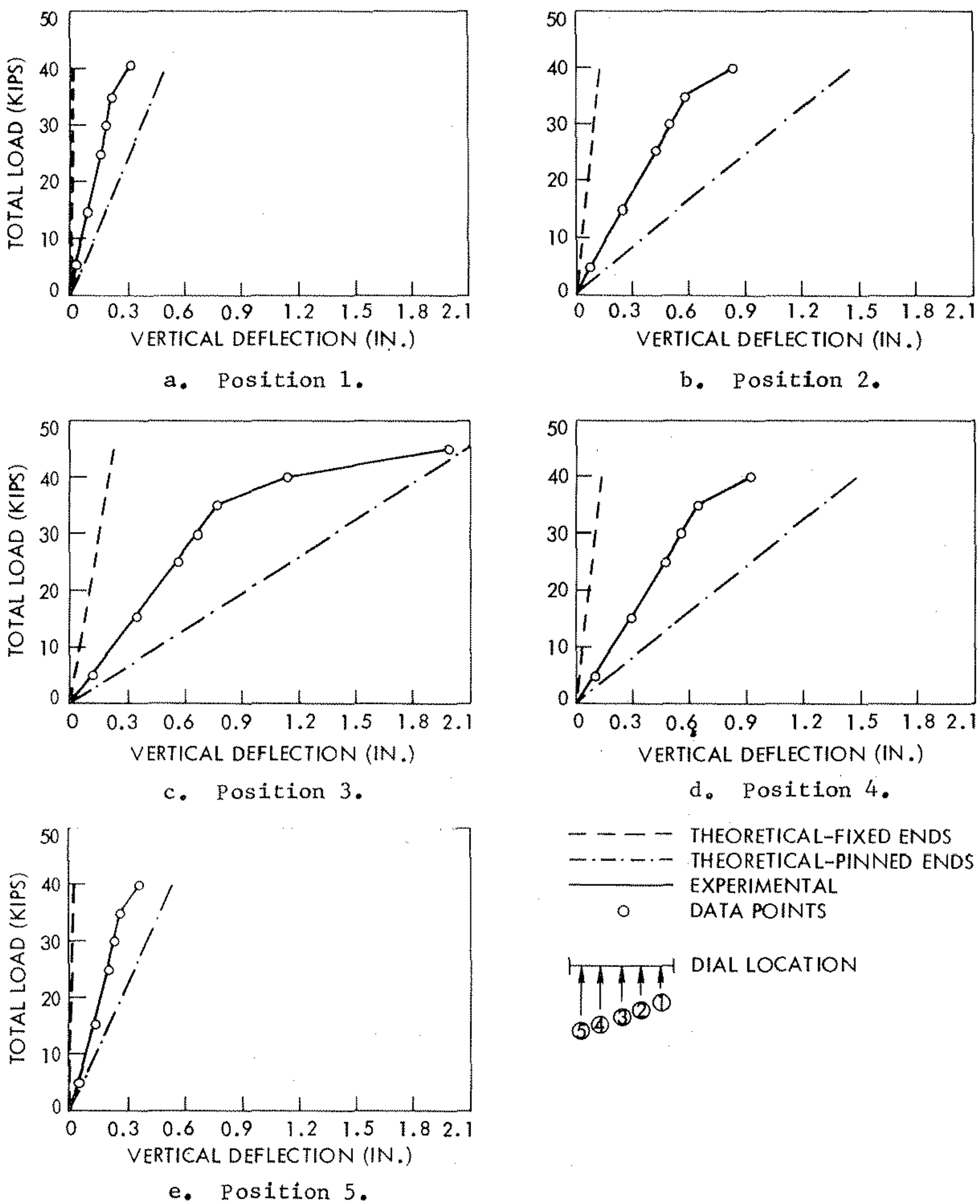
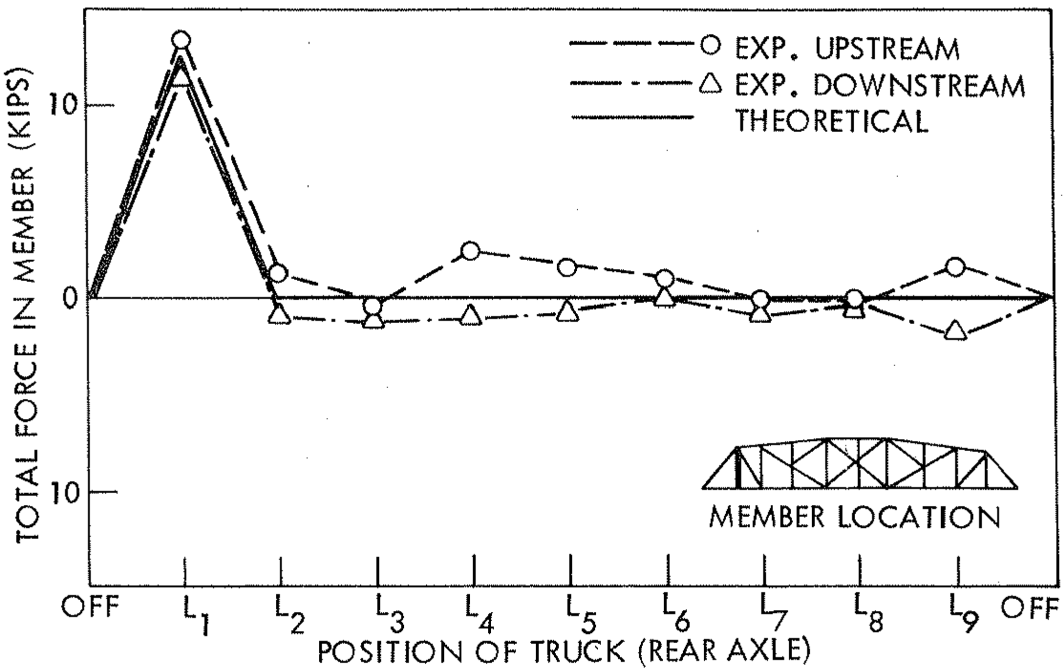
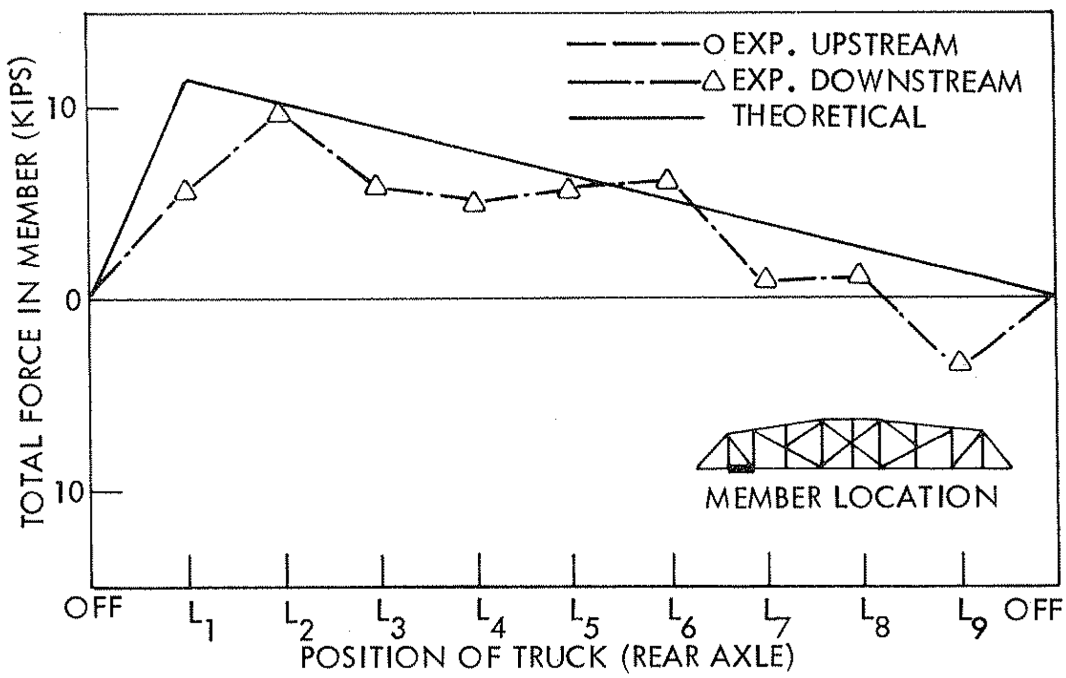


Fig. 29. Load-deflection for floorbeam test at L_5.

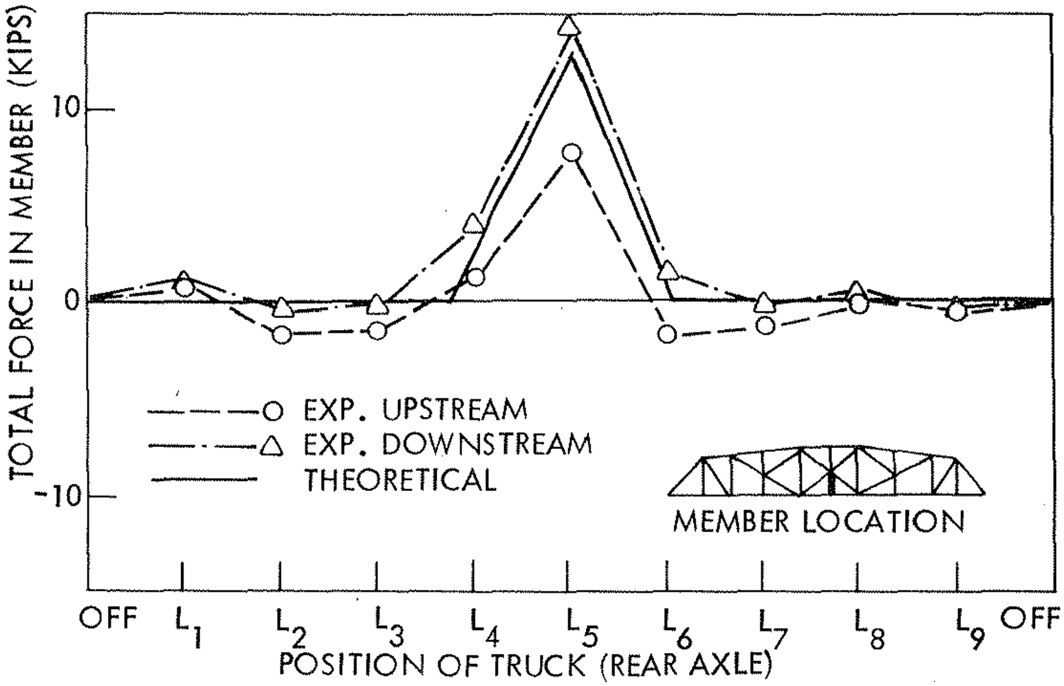


a. Member L_1U_1.

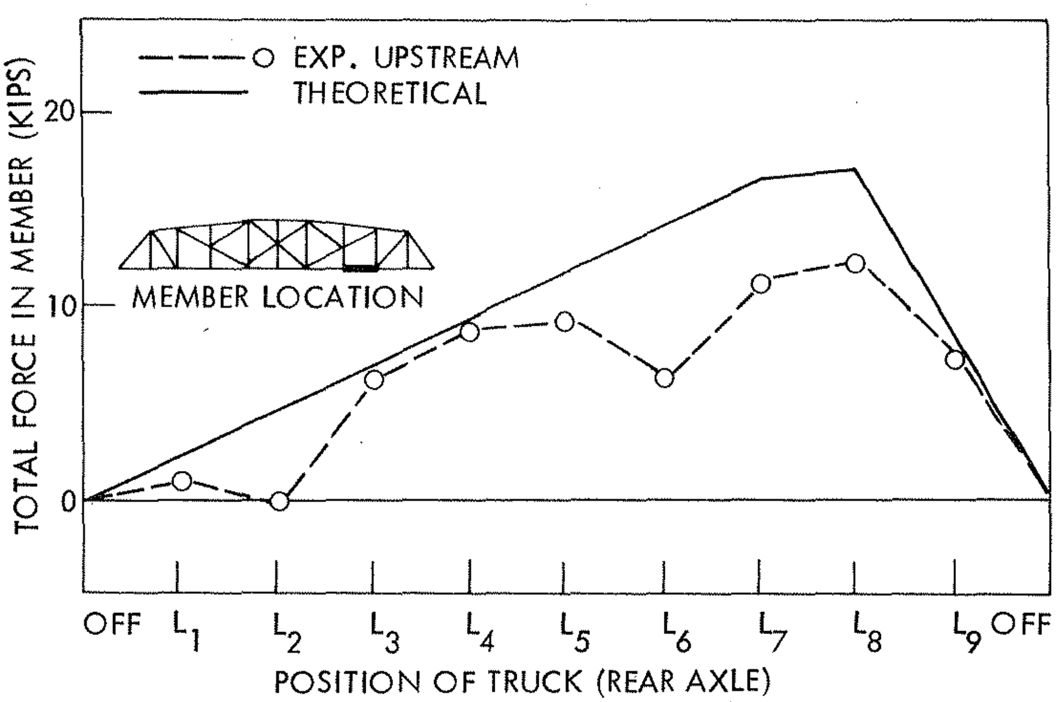


b. Member L_1L_2.

Fig. 30. Influence lines - Span 1, Hubby Bridge.

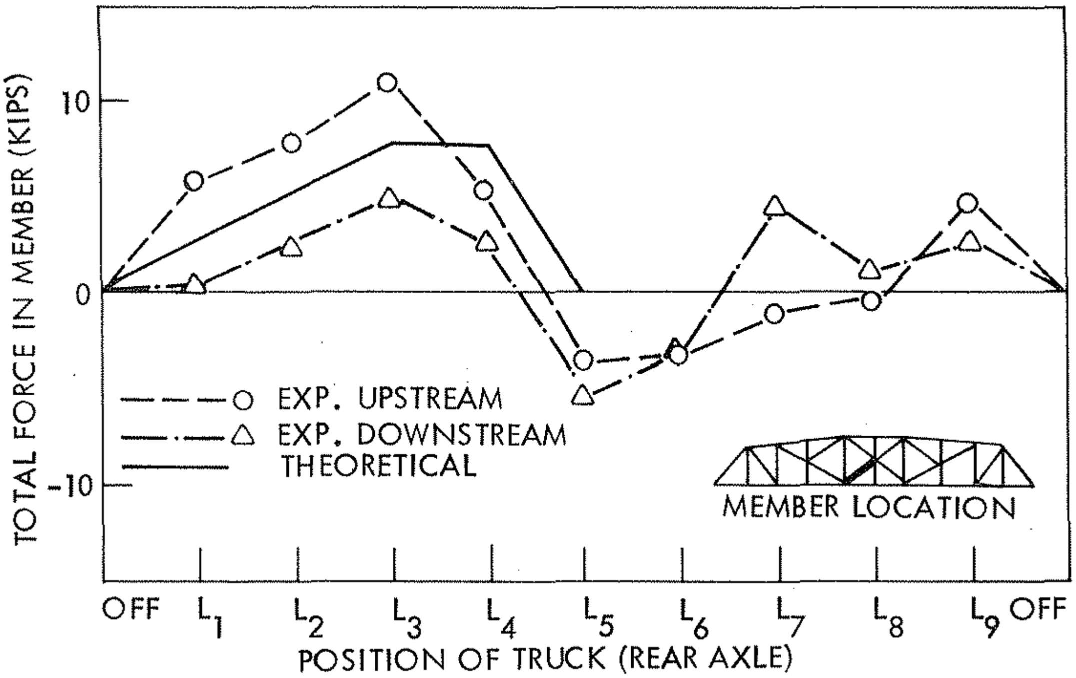


c. Member L_5M_5.

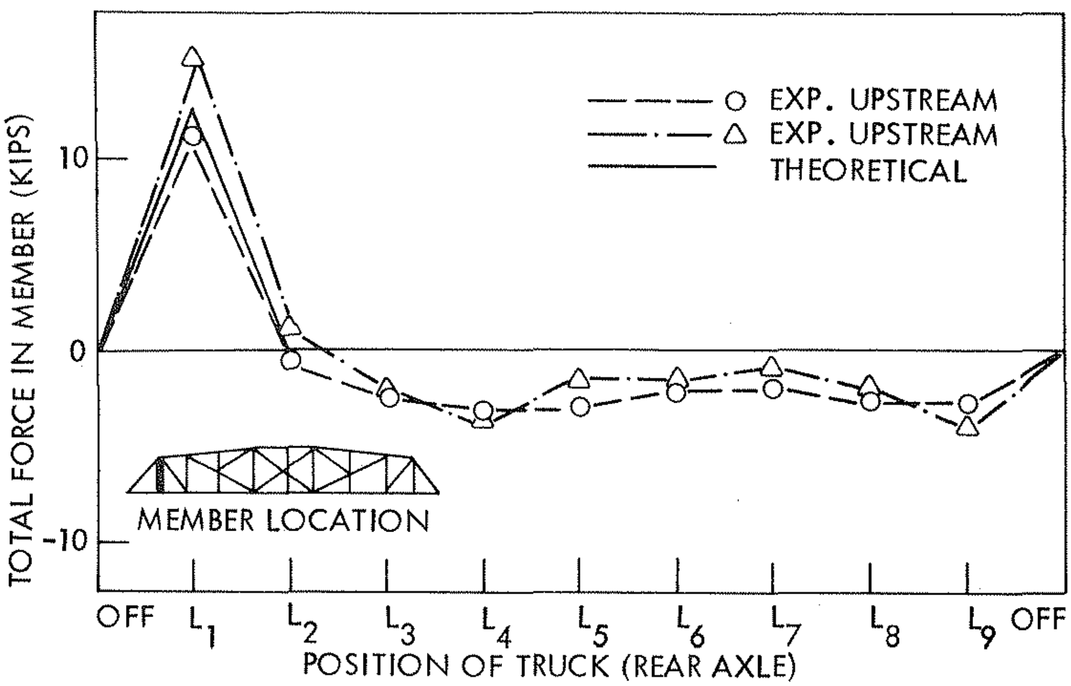


d. Member L_7L_8.

Fig. 30. Cont.

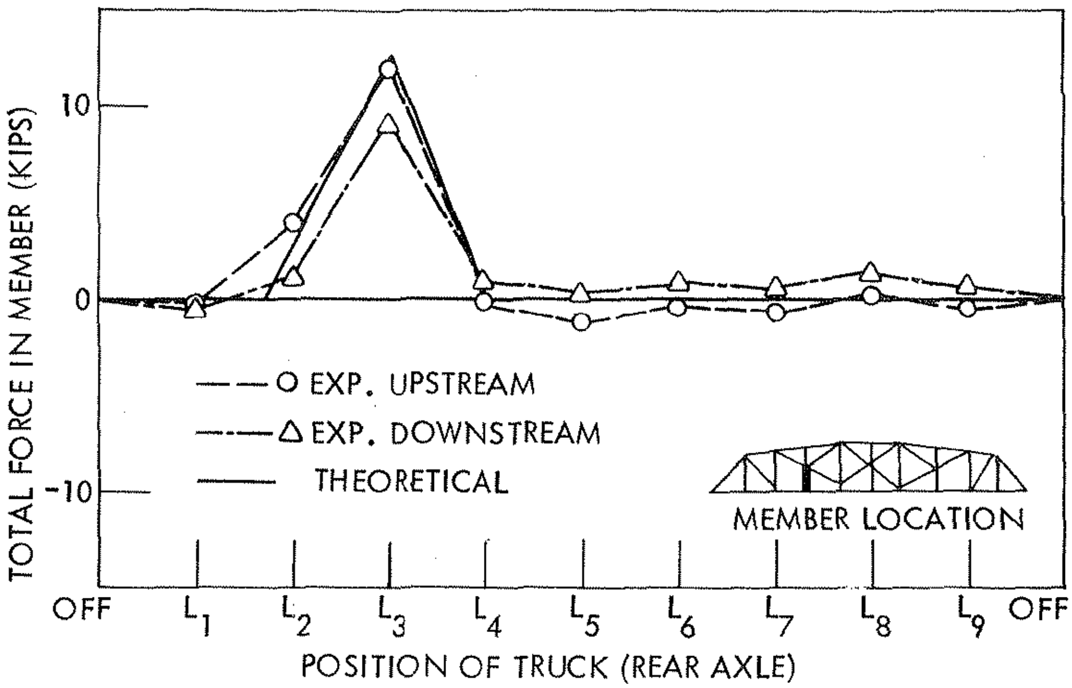


a. Member L_4M_5.

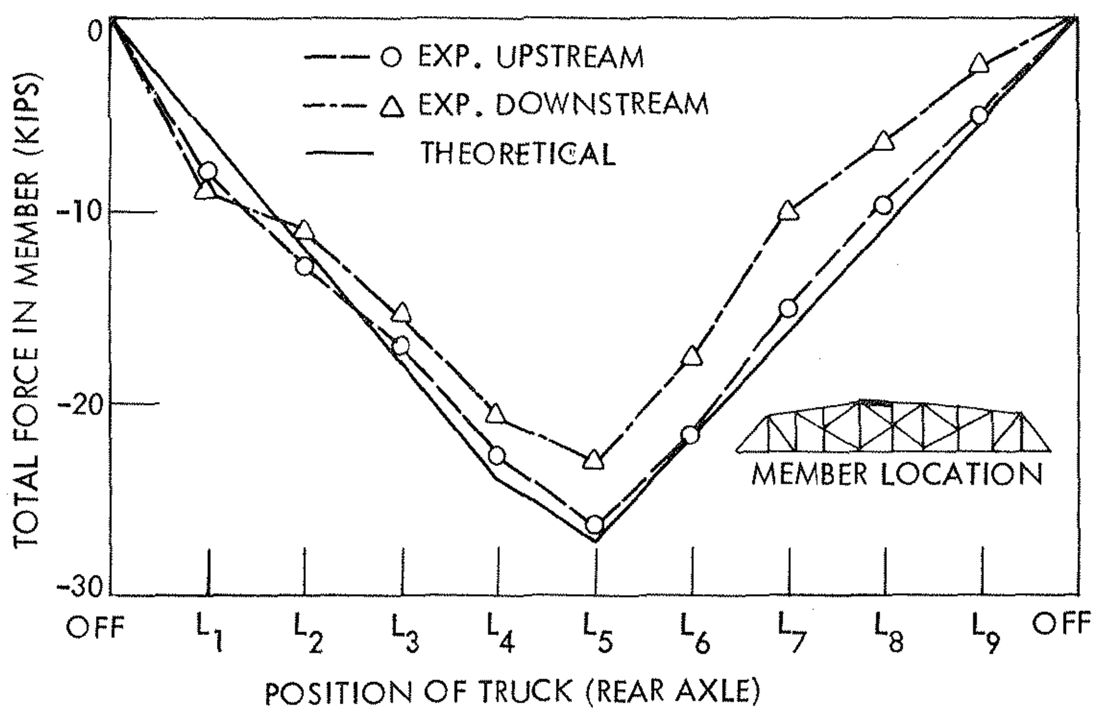


b. Member L_1U_1.

Fig. 31. Influence lines - Span 2, Hubby Bridge.

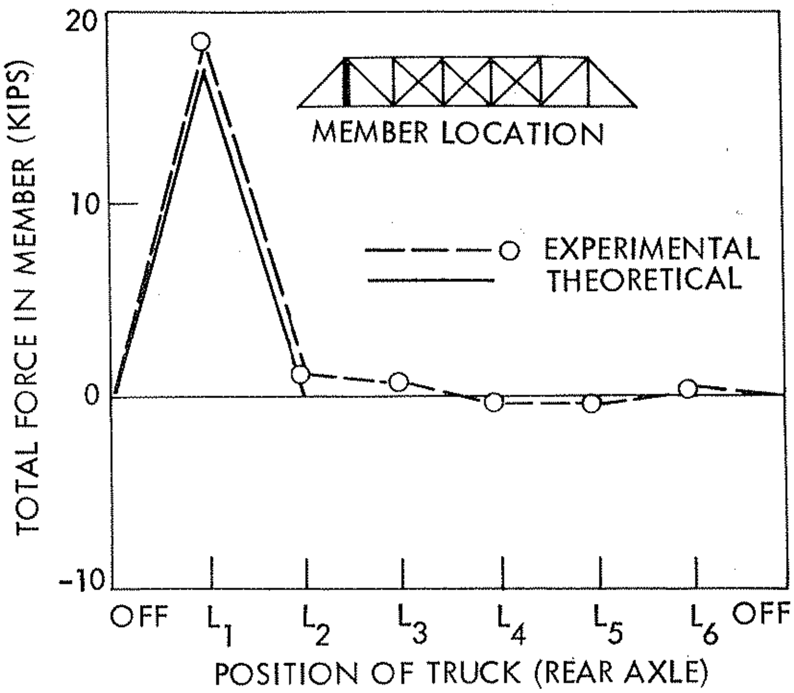


c. Member L_3M_3.

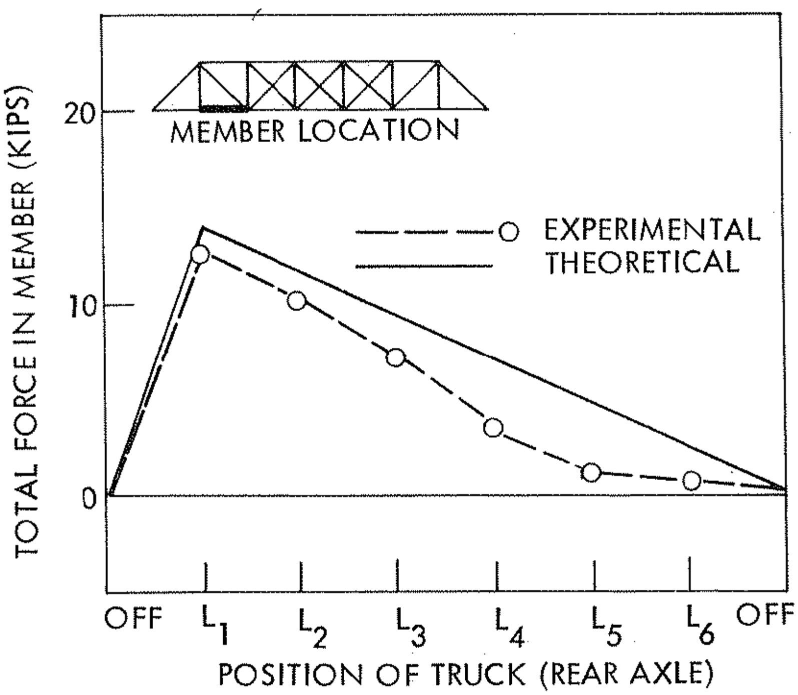


d. Member U_4U_5.

Fig. 31. Cont.



a. Member L_1U_1.



b. Member L_1L_2.

Fig. 32. Influence lines - Chestnut Ford Bridge, truck 2' from left edge.

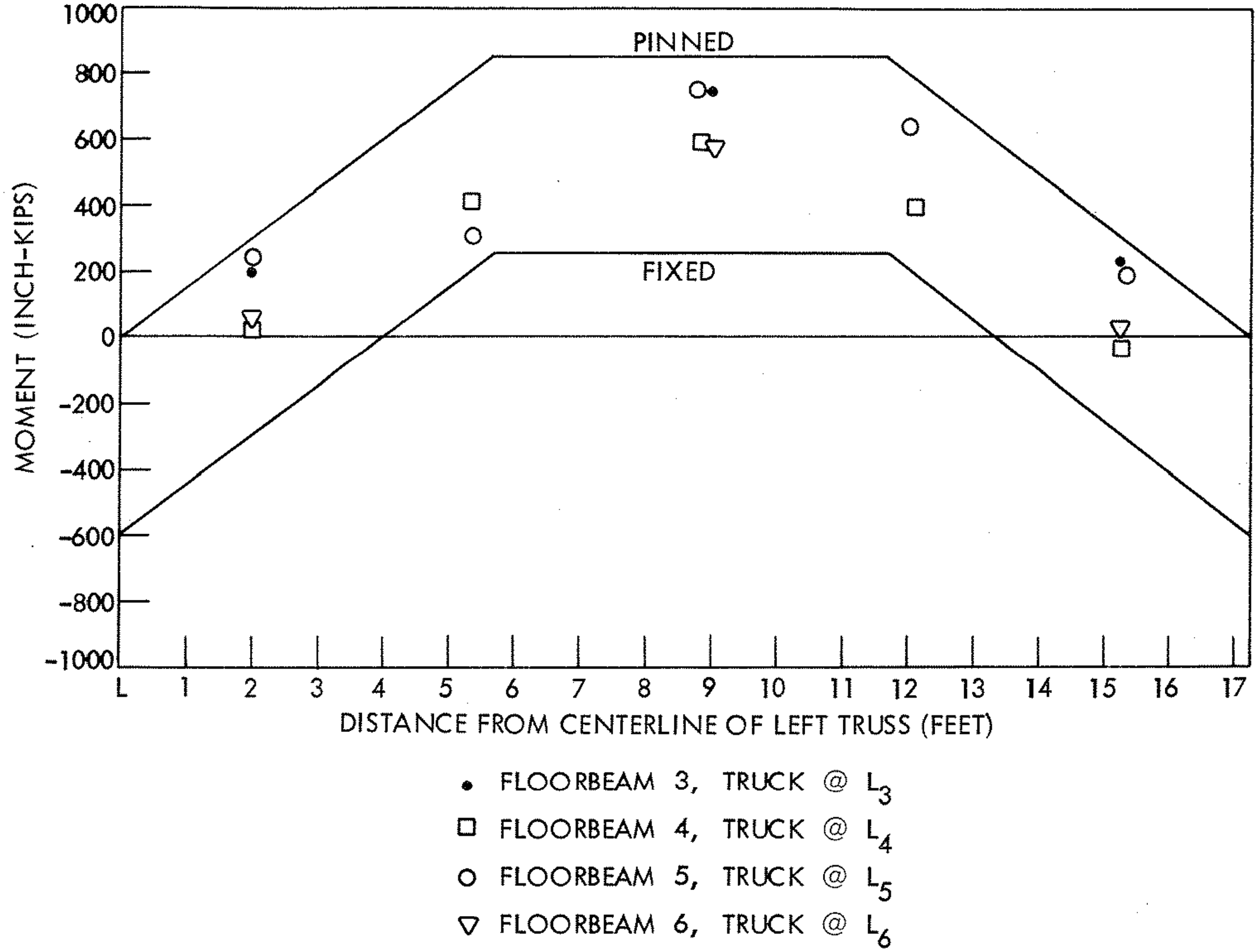


Fig. 33a. Moment for floorbeams at L_3, L_4, L_5, L_6 - truck on centerline.

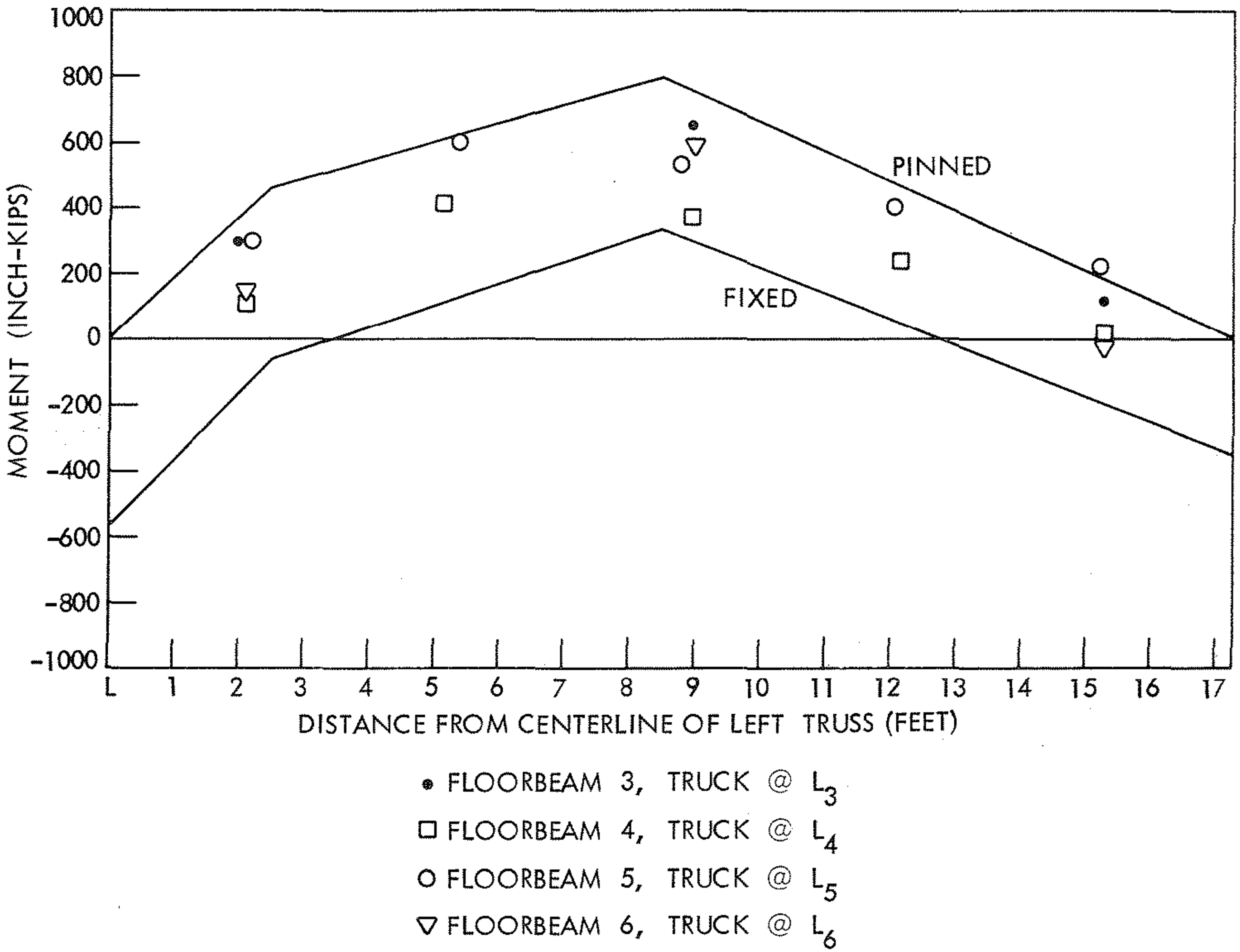


Fig. 33b. Moment for floorbeams at L_3, L_4, L_5, L_6 - truck on left edge.

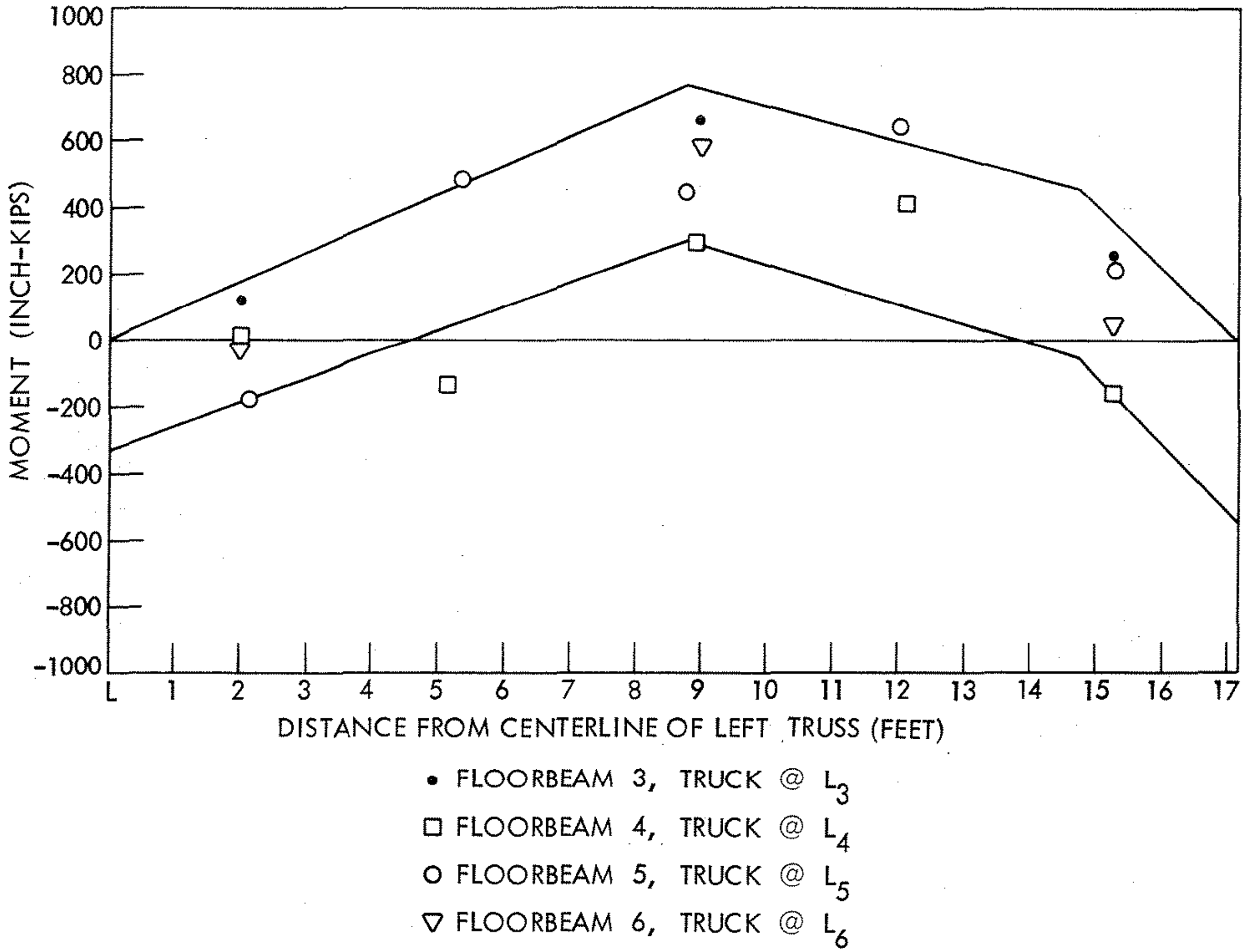


Fig. 33c. Moment for floorbeams at L_3, L_4, L_5, L_6 - truck on right edge.